Structural Stability of Low-Crested Breakwaters

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Structural Stability of Low-crested Breakwaters

by

Morten Kramer

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Preface

The present thesis has been submitted as one of the requirements for obtaining the PhD degree. The study was carried out 2002-2005 at Aalborg University (AAU), Denmark. Professor Hans F. Burcharth, Department of Civil Engineering, Aalborg University, acted as supervisor during the study. Most of the work was carried out in the framework of the DELOS project funded by the EU under contract EVK3-2000-0041.

The text is not intended to replace any aspect of existing design references, but rather to complement them by giving new design guidance on structural design which is specially related to design of low-crested structures. Due to the nature of the research the guidance cannot be considered comprehensive, but it is hoped that the new design formulae and guidance for structural stability will advance present best practice.

The thesis is structured as a main part starting with *1 Introduction* and ending with *14 Notation*. Details and further information is included in appendices numbered *A* to *H*.

I wish to thank the all participants in DELOS for motivating and interesting discussions during project meetings. The technical staff at the Hydraulics and Coastal Engineering Laboratory is gratefully acknowledged for help on setting up experiments in the laboratory. I would like express my gratitude to my colleagues within the department for useful discussions and assistance during the study.

Last but not least I would like to thank my wife and children for their support and patience throughout the period.

Morten Kramer, December 2005

Abstract

A more and more widespread way to protect the coast against ongoing erosion is to build so-called Low Crested Structures (LCS's). Despite a large number of coast parallel LCS's exist, the structural performance of these structures are not fully clarified. The LCS's dealt with are coast parallel detached rubble mound structures, either emerging slightly above the water surface or somewhat submerged like a reef.

Initially results of a study of the geometry of existing LCS's are presented. The geometry and structural performance of existing LCS's forms the basis of the limits for new design equations. New improved design formulae for calculation of static stability of LCS's are developed on the basis of new 2D and 3D laboratory experiments with scale models. The formulae are specially designed for breakwaters subject to shallow water waves and/or depth limited waves, as the majority of existing LCS's are exposed to such conditions. The formulae are validated against prototype experience. Ecological aspects in relation to structural stability are important, and design guidance on how to consider ecology in the design is therefore given. The new design guidance adds practical and helpful knowledge to the toolbox of the designing engineer.

Keywords: Breakwaters, coast protection, low crested structures, model tests, stability.

Contents

Contents in detail

1 Introduction

Long stretches along the European coastline are threatened by erosion. At the same time the economical importance of these areas is rising caused by increased population concentration. This has resulted in comprehensive coastal protection works all over Europe. An often-used coastal protection type is coast parallel breakwaters typically constructed as rubble mound structures. In Denmark it has until now been practice to built such structures with freeboard heights (vertical range between mean sea level and the crest of the structures) of approximately 1.5 to 2.0 metres. These structures are seldom overtopped by waves. Structures with small freeboard heights, Low Crest Structures (LCS's), are often overtopped. In relation to shore protection such structures are by scientists esteemed to be efficient and economically attractive.

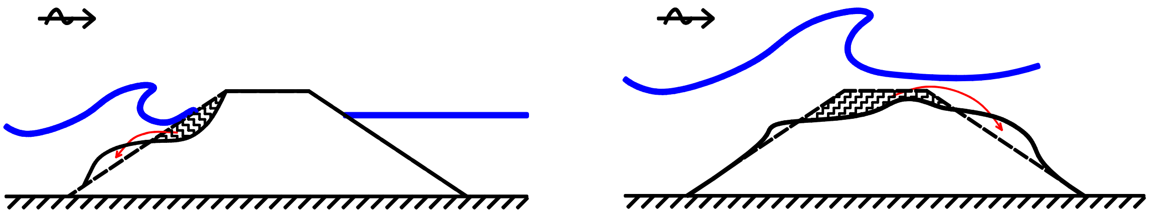
In the recent years the awareness in the population regarding environmental impact related to design of structures is playing an important role. LCS’s are expected to be more suitable/less damaging to the environment than traditional breakwaters, for instance regarding the water quality behind the structures. Moreover due to the low crest height the LCS’s are visually more attractive.

Figure 1.1. Photo of the breakwaters at Lønstrup, Denmark. Photo by Morten Kramer.

The breakwaters on Figure 1.1 are parallel to the coast line, and were built to protect the coast from erosion. The photo was taken at summertime in calm conditions and therefore the breakwaters are not low-crested. However during storms in the autumn the water level rises and these breakwaters becomes significantly overtopped. Waves are breaking offshore of the structures even though the waves are very small. This indicates that the breakwaters are built in very shallow water, which is very often the case for these kinds of structures.

A breakwater with a certain height might be termed “conventional” during normal water levels and wave conditions. However, during high tide and/or storm surge the same breakwater may experience significant overtopping and may therefore be termed “low-crested” as shown in Figure 1.2. For conventional breakwaters only a small amount of energy is allowed to pass over or through the structure. Damage will therefore mainly happen to the front slope. For the low-crested structure wave energy can pass over the structure and dissipate on a larger area of the structure. The low-crested structure is therefore more stable than the conventional type, and smaller rubble stones can be used in the armour layer.

Conventional breakwater | **Low-crested breakwater**



Conventional breakwater:

- *Low wave transmission*
- *High wave reflection*
- *Little or no overtopping*
- *Damage mainly to front slope*

Low-crested breakwater:

- *High wave transmission*
- *Low wave reflection*
- *Significant overtopping*
- *Damage to front, crest and rear*

Figure 1.2. Characteristics of conventional and low-crested breakwaters.

The thesis covers shore-parallel low-crested and submerged structures such as regularly overtopped emergent and submerging detached breakwaters. LCS's can be constructed as a single structure (Figure 1.3a) or in series (Figure 1.3b). A single structure is used to protect a localized area, whereas a multiple segmented system is designed to protect an extended length of shoreline. Submerged breakwaters might be constructed as long continuous structures in which case gaps might not be strictly necessary for water exchange. In schemes with emergent breakwaters or slightly submerged structures such gaps might be provided anyway to allow passage of boats. Figure 1.3c shows an example of a scheme consisting of long submerged breakwaters with small gaps between them. Also shown are some submerged terminal groynes forming a cell configuration often used to retain artificial sand fills.

Single structures as shown in Figure 1.3a are usually built in water depths of more than 3-4 metres with the objective of reducing or stopping coastal erosion at a single location and at the same time creating a sheltered area for swimming or mooring of boats. Detached breakwaters in multi-structure schemes are often constructed in very shallow water of few metres water depth close to the shoreline with the single objective of protecting a beach against erosion and flooding of low-lying areas. If built at some distance from the shoreline the objective would most often be a combination of beach protection and creation of a suitable area for recreational usage.

In general a low-crested breakwater consists of the following parts:

- An outer armour layer of large stones
- A bedding layer of smaller stones and/or geotextile between the bottom of the structure and the sea bed
- Toe protection consisting of armour layer stones or smaller stones

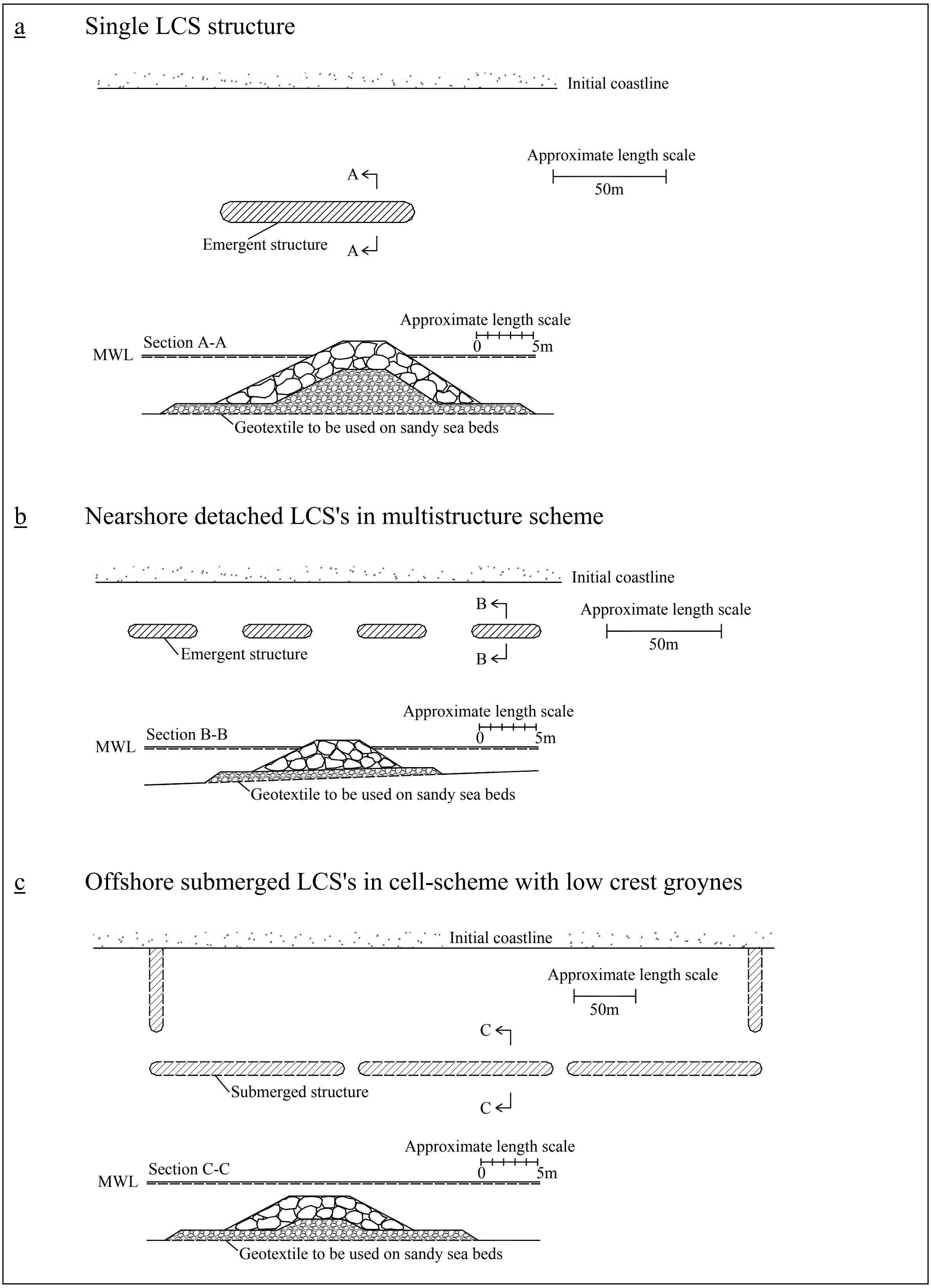


Figure 1.3. Examples of layouts and cross sections of LCS's.

In almost all locations in Europe suitable rock and stone material for LCS's is economically available due to the rather limited costs of long distance shipping materials by barge. However, in many cases nearby land based sources with sufficient quality and sizes exist. Concrete blocks are generally never used for low-crested breakwaters.

The fact that finer rock and stone materials generally are cheaper than larger size materials leads to preference for layered designs instead of more homogeneous designs based on very few sizes or classes of materials. In any case, sufficient filterlayers must be provided between sandy seabed and the coarser structure materials. Geotextiles are often used for this purpose.

For structures of limited height it is not possible to have several layers of different grain/block sizes due to the large size of the armour blocks compared to the total height of the structure. In such cases almost uni-size blocks will be used for the main body resulting in a very permeable structure as opposed to structures with a core of finer materials. In case of deeper water there is a choice between homogeneous structures and layered more impermeable structures. The target wave penetration and exchange of water through the structure then determines the type of design.

A toe protection of a certain width must be provided, usually made flexible by the use of stone and geotextiles to allow for some sea bed scour close to the structure. Toe protection is necessary both on the front and the rear side of the structure.

Various designs of cross-section composition and shape exist. A sketch of a characteristic cross-section built to prevent coastal erosion in Denmark is shown in Figure 1.4. The level of the crest is seen to be 1.3 m above MWL indicating that the structure is not low-crested under normal wave conditions. However, storm surge can be around 1.5m above MWL making the breakwater heavily overtopped. In Figure 1.5 a typical cross-section of a submerged breakwater along the Emilia Romagna coast (North Adriatic coast) in Italy is shown.

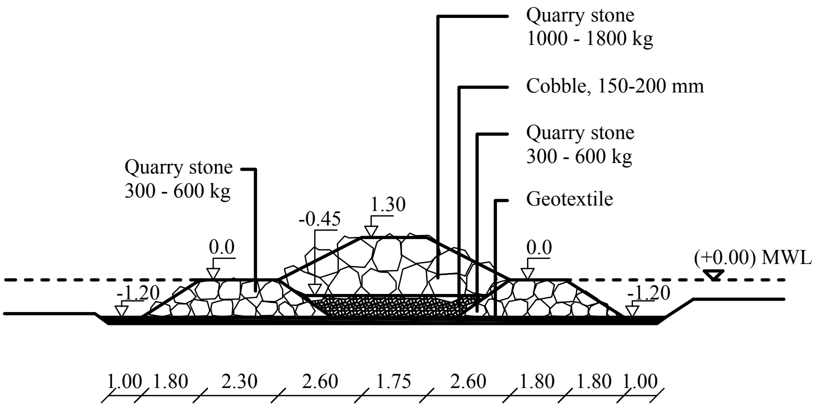


Figure 1.4. Cross-section of breakwaters at Lønstrup, Denmark. Measures in metres. Redrawn after Laustrup and Madsen (1994).

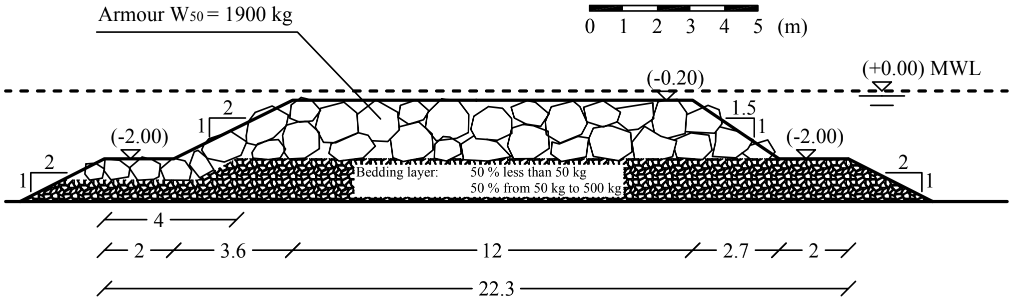


Figure 1.5. Cross-section of a submerged breakwater along Emilia Romagna coast, Italy.

The cross-section shown in Figure 1.4 is narrow-crested and relatively high compared to the submerged wide-crested breakwater in Figure 1.5. Typically also the leeward side of LCS's are exposed to direct wave action due to overtopping waves, and it is therefore necessary to design a toe berm on both sides of the breakwater. If the breakwaters are very high and/or wide, then overtopping will reduce and the toe berm on the leeward side of the breakwater can be designed using smaller stones.

1.1 Overview of design tools related to LCS's

Structural design usually contains a detailed examination of the performance of the various parts of the structure and an economical optimization based on amounts and types of materials, methods of construction, and long-term maintenance. The designer must be in command of a variety of fields, see Figure 1.6. This thesis is focussing on structural design tools with respect to structural stability. The functional design of LCS's is determined by coastal protection performance and hydrodynamic characteristics. These subjects are treated in detail in Burcharth and Lamberti (2006). The lower the crest level of the LCS's, i.e. the larger the wave transmission, the smaller the morphological impact of the structures, which generally means less protective effect. As wave transmission is one of the key design elements a brief introduction to this topic is given in Chapter 10.

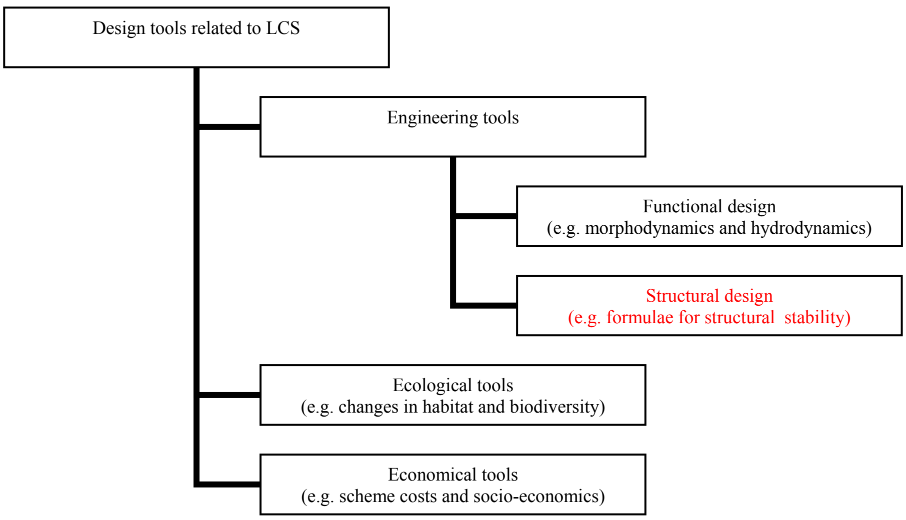


Figure 1.6. Design tools related to LCS.

The final structural design is typically based on experience, numerical and/or parametric models. Simple parametric formulae for structural stability will normally be sufficient for detailed design for LCS's. In case of design of very large structures reference is given to breakwater design tools, for example as given in Chapter IV of the *Coastal Engineering Manual* (Burcharth and Hughes, 2003) and the *Manual on the use of Rock in Hydraulic Engineering* (CIRIA/CUR, 1991). Only limited guidance is available on the design of LCS's. Some recommendations specially related to the design of LCS's can be found e.g. in Crossman *et al.* (2003), Burcharth *et al.* (2006), and Burcharth and Lamberti (2006).

1.2 Model tests with LCS's

Numerical models are still too inaccurate to describe the stability phenomenon especially in case of 3D-waves; therefore numerical models cannot be used in establishment of formulae for structural design. Instead physical model experiments are performed at a small scale, typically 1:50 to 1:10 with respect to prototype LCS's. Often model tests are performed to validate a considered design. For large expensive designs model tests should always be performed in order to optimize the design. For example, stability tests should be performed to determine the required armour unit size when existing stability formulae does not cover the preferred structure geometry, the in situ bathymetry or the type of armour unit.

Laboratory tests are generally more expensive than numerical modelling. However the reliability of physical models is generally much better, so far.

Generally, with scale models only some pre-selected phenomena can be well represented, whereas at the same time, other phenomena may not be reproduced correctly and suffer from scale effects. This is a hardly avoidable penalty for not matching all the scale requirements. If, however, the scale effects are considered to be of minor importance for the phenomena of direct concern for the design of a structure, the scale model may provide accurate information. Scale modelling is however complex and requires sophisticated facilities and experimental set-ups. Care should be taken to perform adequate testing (e.g. wave generation techniques, methods to reduce scale effects, analysis techniques) and to correctly analyse and interpret the results to obtain the required information.

When setting up an experiment one should consider the importance of the following:

1. Scale effects. Typically viscous forces are relatively larger in the model than in the prototype
2. Laboratory effects. Typically the boundaries are different in model and prototype
3. Missing conditions. For example neglecting effects of wind shear stresses acting on the free surface, which may lead to neglecting generation of waves and circulation currents leeward of the structure.

In order to make ideal set-ups in the laboratory with respect to different subjects one may distinguish between the following types of tests with LCS's:

- **Stability tests** (typically the stable unit sizes of armour, core and toe berm are determined)
- **Hydrodynamic tests** (typically wave transmission and reflection characteristics, overtopping, rip-currents and water level set-up in the lee of the structures are investigated)
- **Morphological tests** (typically scour, beach development, and selection of sand for beach nourishment is studied).

An example of the design of model tests related to LCS's can be found in Kramer *et al.* (2005).

Tests can be performed with either fixed bed (solid boundaries, typically concrete bed) or movable bed (to study sedimentary processes, typically a sandy bed). Some laboratories are specialized in moveable bed tests while others only perform fixed bed experiments. Typically fixed bed tests are cheaper and more easily controllable than moveable bed tests. Therefore usually only morphological tests are performed with moveable bed. In fixed bed tests the bottom bathymetry can be either horizontal, sloping or a certain bathymetry can be modelled e.g.

in concrete. In movable bed tests the bed is typically horizontal at the initiation of the tests. During testing the bed forms and e.g. scour holes develop.

Tests can be performed in wave channels (often referred to as 2D-tests) or in wave basins (often referred to as 3D-tests). Wave channel tests are cheaper than wave basin tests. Phenomena related to perpendicular wave attack on the trunk of the LCS are typically studied in wave channels, while phenomena related to the roundhead and effects of oblique waves and 3-D waves are studied in wave basins.

In order to minimize viscous scale effects the model is typically designed as large as the laboratory limits and the economy permit. If the Reynolds numbers are sufficiently large scaling can be performed solely by Froude's model law. As an example the effect of Reynold numbers on the stability of armour stones have been investigated by various researchers. No scale effects seems present if

$$\text{Reynolds number} = \frac{\sqrt{g \cdot H_s} \cdot D_{n50}}{\nu} > 1.0 \cdot 10^4 \text{ to } 4.0 \cdot 10^4 \qquad \text{Eq (1.1)}$$

where g gravitation acceleration
H_s significant wave height
D_{n50} characteristic stone size
ν kinematic viscosity.

If for example a significant wave height H_s = 0.2 m is generated in the laboratory then a stone size D_{n50} = 0.03 m gives a Reynold number $4.2 \cdot 10^4$ (with typical values of $\nu = 10^{-6}$ m^2/s and g = 10 m/s^2). According to the limits given, no significant viscous scale effect is present, and the scaling can be performed by Froude's law.

In 2D hydraulic model stability tests on LCS's it is very important that the set-up in the leeward side of the structure is well controlled. If not controlled overtopping waves will accumulate water behind the breakwater, which will cause a backward flow over the crest and through the structure if permeable. This effect can influence the damage directly and indirectly by changing the wave breaking on and in front of the structure. Thus it should be made clear for which set-up levels the model tests are performed. In 3D test in wave basins the set-up is usually negligible due to the unhindered return flow around the heads.

For a comprehensive study of physical models and laboratory techniques, see Hughes (1993).

1.3 Utilization of the thesis

The design of LCS's is diverse throughout the world. In order to clarify this subject the author collected information from several EU countries about the geometry of existing LCS's. The outcome of this investigation is described in the following chapter. In several existing schemes with LCS's problems with structural stability has resulted in reduced performances and costly maintenance. Only very limited formulae and recommendations for structural design of LCS's exist. Adequate stability formulae and guidelines on design waves and water levels have proved to be insufficient. Based on new 2D and 3D model tests the author has produced two new armour layer stability formulae, and guidelines on choosing the proper hydrodynamic conditions for structural design has been derived. The recommendations and formulae have been developed focussing on getting simple and practical design tools.

2 Layout of existing structures

The geometry and layout of existing LCS's are investigated with the purpose of describing characteristics of European LCS's in respect of a worldwide scenario. An interesting statistical study for such structures in Japan can be found in Takaaki (1988) and parameters for structures in USA can be found in Chasten *et al.* (1993) and McIntosh and Anglin (1988). For European structures no literature containing statistical information exists, but within DELOS an inventory on physical engineering properties of LCS's has been established, see Appendix A. For further information regarding the DELOS databank, see Kramer (2002) and Lamberti *et al.* (2005). Only the main conclusions from the study is included in the following.

The data in the European inventory was collected from seven EU countries aiming at representing a broad range of structural layouts. The inventory data were organized in a data bank assembled from 150 completed questionnaires. Each completed questionnaire contained a scheme containing several structures often of various types, e.g. a system of segmented offshore breakwaters with groins closing the scheme in each end. The main purpose of most schemes containing low crested structures in the inventory is built for beach and land protection against erosion. A few structures in the inventory are built mainly for coastal protection for ecological reasons or for protection of harbours, inlets, outlets, channels etc.

The typical type of coastal protection scheme with LCS's is to use detached breakwaters (66% of the schemes, see Figure 2.1). In 22% of the schemes a combination of detached breakwaters and groins is used. The detached breakwaters are described by Figure 2.2.

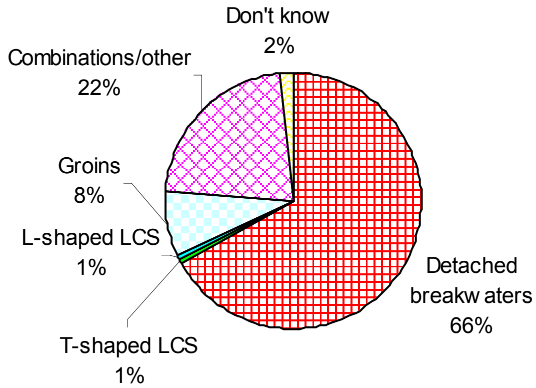
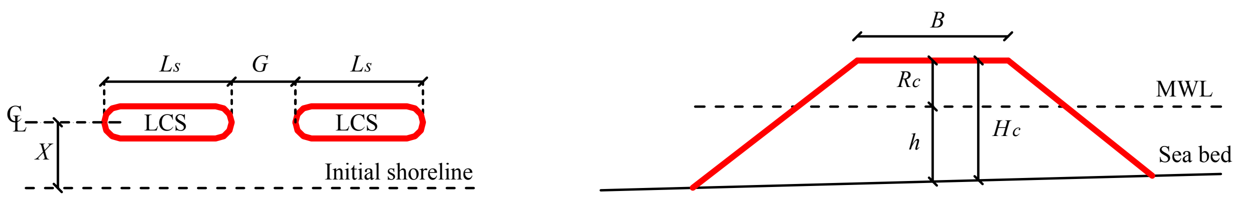


Figure 2.1. Types of low crested structures in Europe, Kramer (2002).



X	Distance between the shore line pre-project and the centre line of the LCS
L_s	Typical length of the segments at crest level
G	Length of the gaps between the structures at crest level
B	Width of LCS at crest level
R_c	Freeboard, the distance from crest level to MWL (negative if submerged)
h	Water depth at MWL

The height of the structure is: $H_c = h + R_c$.

Figure 2.2. Description of structural parameters.

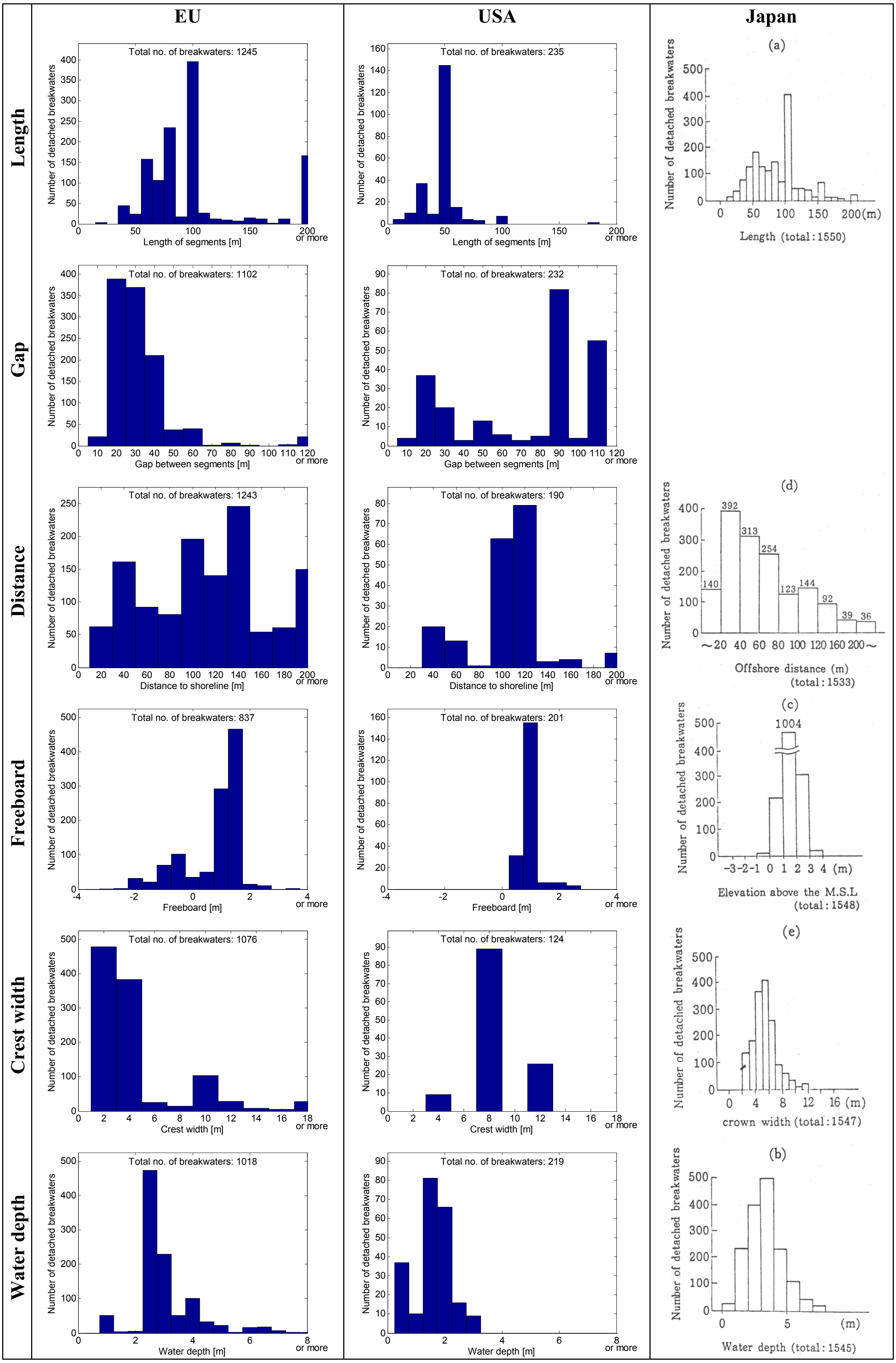


Figure 2.3. Geometry of low-crested breakwaters, graphs showing Japanese layout are by Takaaki (1988).

The total number of investigated breakwaters in EU was about 1200 and in Japan 1550, giving about the same statistical uncertainty in Figure 2.3. The data from US contains data from only 24 schemes containing 235 breakwaters giving large statistical uncertainties in the figures.

In general the same trends for the parameters in Figure 2.3 are found for EU, USA and Japanese structures. However in USA no long (L_s > 100 m) structures exist and some systems are built with large gaps (G > 50 m). Structures with submerged crests seem to be more present in Europe than in Japan and USA. It is seen from Figure 2.3 that the variations in the parameters are large. However typical values exist, e.g. the length of the segments at crest level is about 60 to 100 m with an average value of about 80 m. This leads to a typical layout of the structures. From Figure 2.3 it is also seen, that in some of the histograms two peaks are present. This indicates that two very different cross sections exist; a narrow crested emerged structure and a wide crested submerged type. The typical sets of parameters are shown in Figure 2.4. The typical layouts corresponds to the two examples given in the introduction, cf. Figure 1.4 and Figure 1.5.

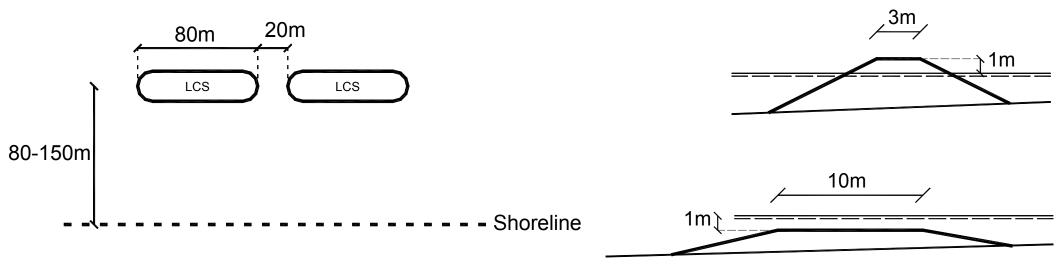


Figure 2.4. Typical structural layout and cross sections.

The inventory has been used within DELOS for many applications, e.g. including structural performance, coastal protection performance, ecological aspects and socio-economic characteristics. In Appendix A more detailed statistical analysis of the geometry of the structures is performed, together with a study of ranges of structural ratios.

The ranges for the structural geometry shown in Figure 2.3 has been used directly to decide on the test ranges for the armour layer stability formulae. The range of freeboard is -2 m to +2 m indicating that the breakwaters in the inventory are indeed low-crested. As the main part of the existing breakwaters are relatively narrow crested (B < 8 m) it was decided to limit the investigation regarding armour layer stability to fairly narrow structures. Another important outcome of the analysis is that almost all low-crested breakwaters are built in very shallow water of depths typically less than five metres. At such water depths the largest waves will be depth limited. As breaking waves are more damaging to the structure than non-breaking waves it is important that formulae for structural stability of LCS's are developed for shallow water conditions.

3 Structural stability in general

A LCS can undergo various structural problems such as deterioration, breakage, settlements, instabilities etc. For LCS's traditionally rock of proper quality is used in the armour layer and sufficiently large armour blocks are applied to ensure static stability. For this reason problems with breakage and deterioration are avoided. Settlements of the structure due to instabilities in the subsoil must be avoided by ensuring proper foundation of the structure, see Figure 3.1.

In the present chapter armour layer stability is examined and in the following chapters toe-berm stability and sea bed scour is briefly discussed.

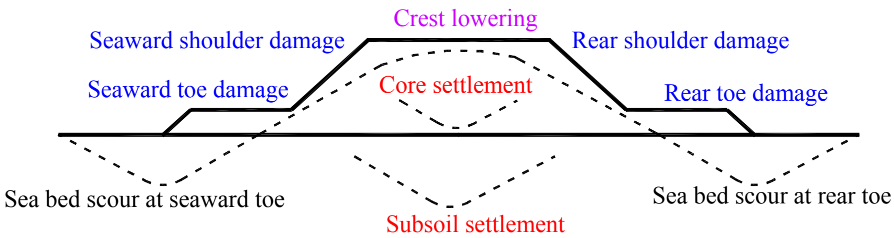


Figure 3.1. Possible problems caused by instabilities or damages to a LCS.

Wave forces acting on a rubble-mound slope can cause armour unit movement. This is called hydraulic instability. Armour unit movements can be rocking, displacement of units out of the armour layer, sliding of a blanket of armour units, and settlement due to compaction of the armour layer. Figure 3.2 shows the most typical armour layer failure modes.

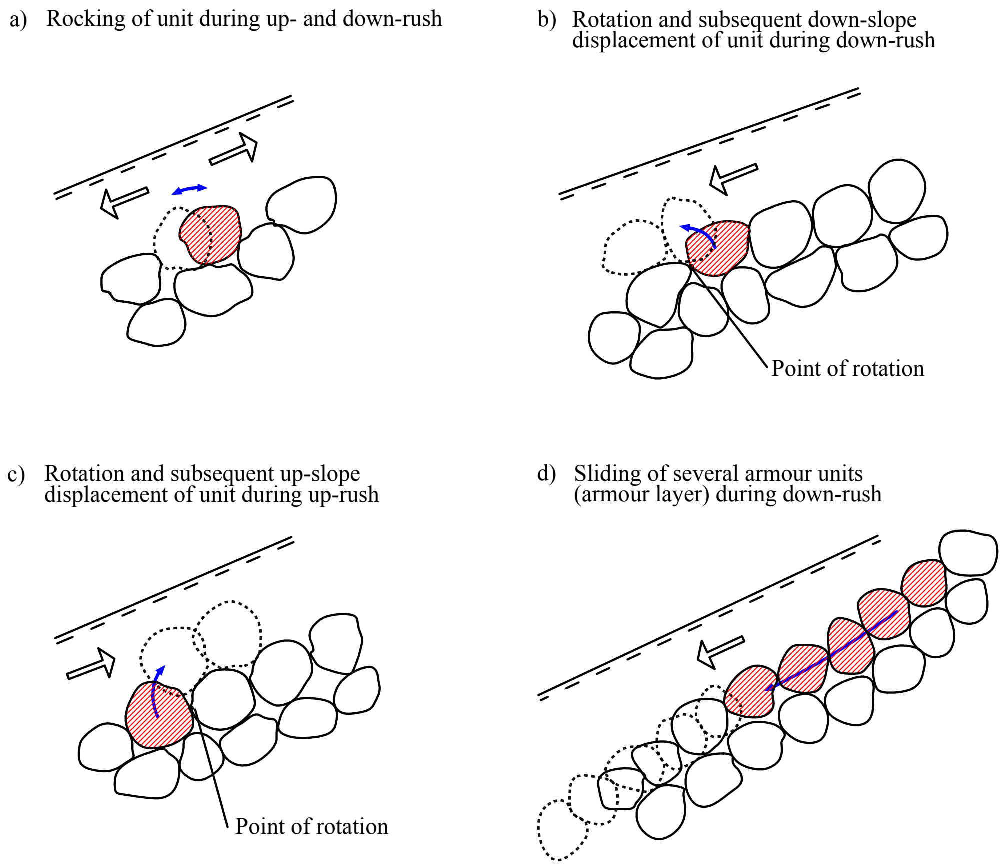


Figure 3.2. Typical armour layer failure modes (Burchart, 1993).

The response of the armour units in terms of movements are related directly to parameters of the incident waves, while treating the actual forces as a “black box” transfer function. However, some qualitative considerations of the involved forces can be used to explore the structure of stability formulae. In the following a brief introduction to parameters used in stability formulae is included. Parts of the text given in Burcharth and Hughes (2003) has been used.

3.1 Stability parameters and structure of stability formulae

The wave-generated flow forces on armour units might be expressed by a Morison equation containing a drag force F_D, a lift force F_L and an inertia force F_I. The stabilizing force is the gravitational force F_G. Assuming that at the stage of instability drag and lift force dominates the inertia force, a qualitative stability ratio can be formulated as the drag force plus the lift force divided by the gravity force:

$$\frac{F_D + F_L}{F_G} \approx \frac{\rho_w D_n^2 V_f^2}{g(\rho_s - \rho_w) D_n^3} = \frac{V_f^2}{g \Delta D_n} \qquad \text{Eq (3.1)}$$

where D_n = (armor unit volume)$^{1/3}$ is the equivalent cube length, ρ_s and ρ_w are the mass densities of armor units and water, respectively, and V_f is a characteristic flow velocity. By inserting $V_f \cong (gH)^{1/2}$ for a breaking wave height of H in Eq (3.1) the following stability parameter, N_s, is obtained.

$$N_s = \frac{H}{\Delta D_n} \qquad \text{Eq (3.2)}$$

where Δ= (ρ_s/ρ_w - 1). Non-exceedence of instability, or a certain degree of damage, can e.g. be expressed in the general form

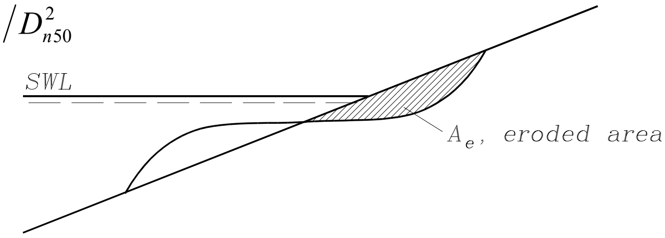
$$N_s = \frac{H}{\Delta D_n} \leq K_1^a K_2^b K_3^c \ldots \qquad \text{Eq (3.3)}$$

where the factors depend on all the other parameters (sea state and structural parameters), except H, Δ and D_n, influencing the stability.

3.2 Overview of damage parameters

Damage to armour layers is characterized either by counting the number of displaced units or by measurement of the eroded surface profile of the armour slope. In both cases the damage is related to a specific sea state of specified duration. The most commonly used definitions of damage parameters are given in Table 3.1.

Table 3.1. Commonly used definitions of damage.

1)	Relative displacement within an area	$D_R = \dfrac{\text{number of displaced units}}{\text{total number of units within the reference area}}$ Displacement has to be defined, e.g. as position shifted more than distance D_n, or displacements out of the armour layer. The reference area has to be defined, e.g. as the complete armour area, or as the area between two levels of armour displacements, e.g. SWL ± H_s, or SWL ± $n_s D_n$, where n_s is a number of stones.
2)	Number of displaced units within a strip with width D_n (van der Meer 1988)	$N_{od} = \dfrac{\text{number of units displaced out of the armour layer}}{\text{width of tested section} / D_n}$
3)	Number of displaced units within total height of armour layer (van der Meer 1988)	N_{od} / N_a , where N_a is the total number of units within a strip of horizontal width D_n
4)	Procent erosion of original cross-section area (Hudson 1958)	$D\% = \dfrac{\text{average eroded area from profile}}{\text{area of average original profile}}$ x 100%
5)	Relative eroded area (Broderick 1983)	$S = A_e / D_{n50}^2$ 

3.3 Trunk armour damage parameter for present tests

In order to compare observed damage given as number of displaced armour stones N, to the Broderick parameter $S = A_e / D_{n50}^2$ for trunk damage, a link between N and S must be established. The eroded volume in a trunk test section is $V_e = N \cdot D_{n50}^3 / (1-n)$, where n is the porosity of the armour layer. As $A_e = V_e / Y$, where Y is the width of the trunk tests section, we obtain

$$S = \frac{N \cdot D_{n50}}{(1-n)Y} \qquad \text{Eq (3.4)}$$

In the new 3-D tests at AAU explained subsequent the following parameters were used: $n = 0.44$, $Y = 0.50$ m and $D_{n50} = 0.0325$ m for the armour in the trunk section. From Eq (3.4) we obtain the simple relationship $S = 0.11 \cdot N$.

3.4 Roundhead armour damage parameter for present tests

To characterize the roundhead damage, the method by Vidal *et al.* (1995) is adopted. They observed, as was seen in the new 3-D tests at Aalborg University, that the region most prone to damage was between levels SWL + H_s / 2 and SWL – H_s and suggested that the reference width for damage quantification is calculated as the arch length $R\theta$, where R is the mean of the head radii corresponding to the two levels, and θ is the angle of actual sector of the roundhead, e.g. for a 60° sector $\theta = \pi/3$.

For roundhead damage we then obtain similar to Eq (3.4)

$$S_{head} = \frac{N \cdot D_{n50}}{(1-n)R \cdot \theta}$$

where

$$R = \frac{B}{2} + \begin{cases} \cot\alpha\left(H_s + R_c\right)/2 & , R_c \leq \dfrac{H_s}{2} \\ \cot\alpha\left(H_s/4 + R_c\right) & , R_c > \dfrac{H_s}{2} \end{cases} \qquad \text{Eq (3.5)}$$

α is the slope angle

3.5 Defining the degree of acceptable damage

The degree of damage for conventional breakwaters is traditionally categorized as follows (according to definitions by Losada *et al.*, 1986):

ND: No damage (maybe one or two loose stones starts rotating)
ID: Initiation of damage (a few stones starts to move)
IR: Iribarren damage (big holes in the outer armour layer, but the filter layer is not visible).
D: Destruction (filter layer is exposed to direct wave attack)

As low-crested structures are built in shallow water the highest waves will often be depth limited. The structures will typically be exposed to design waves numerous times during the lifetime. As damage is cumulative it is important to design such structures for a low damage criterion. Moreover for ecological reasons, as described in detail in Chapter 4, structures should at all times be stable requiring a minimum of maintenance. Design recommendations and results given subsequent are therefore given for initiation of damage.

For the trunk in the new 3D experiments a uniform distribution of S was chosen corresponding to initiation of damage such that $S = 0.5$ for the seaward slope, $S = 0.5$ for the crest, and $S = 0.5$ for the leeward slope. $S = 0.5$ corresponds to approximately 4 displaced stones along the 50 cm wide test section in the experiments. The choice of S corresponding to "initiation of damage" were based on visual observations of the damage in the experiments and the exposed areas shown in Table 5.3. Some authors, e.g. Van der Meer *et al.* (1996) and Vidal *et al.* (1995) have performed similar experiments and suggest a higher S-value for the seaward slope than for the remaining parts of the trunk. The new AAU experiments did not show any behaviour of the eroded area to support a larger S-value for the front slope.

For the roundhead a uniform distribution was chosen, such that $S = 1$ for the seaward head, $S = 1$ for the middle head, and $S = 1$ for the leeward head. Again there were no reasons to allow larger S-values in some regions.

3.6 Parameters influencing armour layer stability

The necessary armour rock size to ensure structural stability is in general influenced by the degree of acceptable damage and physical characteristics such as structural parameters, materials and hydrodynamics. The damage parameters used for the quantification of the damage is of course closely related to the degree of acceptable damage, but is not directly influencing the stability. Stability formulae developed for a certain degree of damage is therefore not dependent on the damage parameters used for establishing the formulae.

The stability of a structure may in general possibly be influenced e.g. by:

Surrounding seabed
- Type of seabed (sandy, rocky, ...)
- Bathymetry/foreshore slope
- Existence of river outlets, harbours etc.
- Sediment transport (direction, amount and distribution over the coastal profile)

Structural outer shape
- Crest width
- Structure height
- Freeboard
- Structure slope
- Geometry of roundhead
- Gaps between structures

Characteristics of materials
- Type of units
- Size and density of units
- Shape and grading
- Thickness of layers
- Packing density (porosity) and construction method
- Core and filter material sizes, types, shape/grading, layers, porosity

Hydrodynamic parameters
- Wave height (characteristic values and/or distribution)
- Wave breaking (non-breaking, breaking and broken)
- Wave steepness or wave period
- Main wave direction
- Directional spreading of waves
- Current velocities and directions (characteristic values and/or profiles and distributions)
- Water level variations caused by surge and tide

Many of the above parameters are related and not all parameters are equally important for the stability of LCS's. However, it is not always easy to decide whether a given parameter is important or not. Further detailed parameters covering any aspect are never known in real case applications. Therefore stability formulae should only contain the most important parameters and only parameters which are well known for a specific site or easily obtainable should be included in a formula. Some of the parameters given above are discussed in the following.

Effect of mobile bed

Usually armour stability experiments are carried out with a fixed seabed. On a real mobile bed scour close to the structure will take place leading to possible instabilities for the structure toe. This phenomenon is discussed further in Chapter 9. A large variation in the local bathymetry or a big scour hole close to the structure might however also increase the possibility for locally higher waves, and thereby decrease the local stability of the breakwater. Another phenomenon is that sand intrusion in the structure can make the structure less permeable, which will lead to a less stable structure.

The effects of the surrounding seabed is generally only taken care of in stability formulae through the calculation of the waves. As LCS’s are built at low water depths local changes in bathymetry can be large compared to the water depth. If is therefore important the designing engineer is aware of the influence of the local effects on the stability of LCS’s.

Structural outer shape

All parameters for the structural outer shape are generally important for the stability of a LCS. Especially the freeboard is important for LCS’s as the structures are getting more stable as the freeboard is decreased.

Type and size of armour units

Traditionally rock armour is used for LCS’s. Formulae for amour layer design are usually solely based on model tests with rock armour, and the formulae can therefore only be used for armour layers consisting of quarry rock.

The necessary size of the units is the unknown, which needs to be determined by using the stability formulae. The larger the rock size, the more stable is the structure.

Density of armour units

The density of typical rock is 2650 kg/m^3, but heavier and lighter types exist. By using the traditional stability number $N_s = H_s/(\Delta\ D_{n50})$ in the stability equations, the armour density is included in terms of $\Delta = (\rho_s/\rho_w - 1)$. For the same stability it is hereby assumed that the equivalent cube length is proportional to $1/\Delta$, i.e. $D_{n50} \propto 1/\Delta$, which means high density stone can be smaller ensuring the same stability. This however, is not always true as explained in the following.

Holtzhausen and Zwamborn (1992) performed tests with Dolos blocks with different densities. The tests showed less stability for high density units than predicted. Helgason and Burcharth (2005) performed tests with emerged rubble mound breakwaters with armour rock densities in the range ρ_a = 2650 to 3300 kg/m^3, and concluded that the positive effect of stability from the increased density is overestimated by conventional stability formulae in case of steep slopes. The stability generally depends on structure slope and armour unit type. Further Helgason and Burcharth (2005) concluded that the effect of density is correctly described by traditional stability formulae for rock armour in case of structure slopes 1:2 and most likely for flatter slopes.

For LCS's no specific studies of the influence of armour unit density exist. But, as LCS's are traditionally built with gentle slopes (usually 1:2 or flatter) one may, based on the conclusions by Helgason and Burcharth (2005), presuppose that formulae for LCS's are valid also for high or low density types of rock.

Block shape, grading and porosities

The above parameters are all related to the transmissibility of the structure, which influences the stability. A homogeneous structure with rocks of similar size and shape (i.e. high value of the porosity *n*) is very transmissible for waves and the armour is thereby more stable than for a more impermeable structure. Some formulas, e.g. as the Van der Meer 1990 formula described in Appendix F.2, includes a parameter in the stability formulae for the transmissibility of the structures.

Based on riprap slope tests Thompson and Shuttler (1976) concluded that the shape only had minor importance for the stability, apart from flat and rounded stones which are less stable than angular stones. Further Burger (1995) and Van der Meer *et al.* (1996) investigated the influence of rock shape and grading on the stability of a slightly emerged low-crested breakwater and concluded that the influence was very small, especially for low damage levels. A rock type with relatively many elongated/flat rocks showed a similar stability as more uniform rock types. No influence was found for gradings D_{85}/D_{15} smaller than about 2, but it was recommended not to use gradings with $D_{85}/D_{15} > 2.5$. The conclusion was further to release customary strict restrictions on shape or grading of armour material during construction.

Newbury *et al.* (2002) and Stewart *et al.* (2002) performed an extensive envestigation of influences of armour shape, porosity and placing methods. Newbury *et al.* (2002) concluded: Slope and grading does not affect porosity, placement method affects porosity by 2 - 4 %, rock shape is important for the porosity. Conclusions by Stewart *et al*. (2002) with respect to armour stability: For single layer structures the packing density has little effect, for double layer structures with individually placed tightly packed rocks (i.e. porosity is low < 35 %) the actual stability is higher than predicted. In summary the conclusions means that it is on the safe side to use stability formulae developed by random placement when designing tightly packed structures.

Wave heights, periods and wave breaking

The distribution of wave heights and periods are very important for the stability. Phenomenons such as spectral shape and swell does affect the stability. However these phenomenons are seldom known in detail when the armour layer is designed, and further they are difficult to incorporate in stability formulae. Instead characteristic values are normally used to describe the waves, e.g. as one parameter for wave heights such as the significant wave height H_s, and one parameter to describe the wave periods such as the peak wave period T_p or the peak wave steepness s_p.

LCS's are typically built in shallow water where heavy wave breaking directly on the structure is likely. As breaking waves are more damaging than non-breaking waves it is important to take account for wave breaking in the formulae.

Wave obliquity and directional spreading

Benoit (1995) investigated the stability of conventional breakwaters (not LCS's) in long crested oblique waves (effect of wave direction) and short crested waves (effect of wave direction and angular spreading) and concluded:

- The wave direction does not seem to have a significant effect on the stability of the main armour
- Damage to the main armour is higher under short-crested waves than under long-crested waves for the same incident wave height. Damage is highest for normal incidence 3D waves.

Matsumi *et al.* (2000) performed experiments on non-overtopped breakwaters. Matsumi stated in contrast to other researchers that the armour stability of the heads is lower in case of multidirectional waves.

For conventional non-overtopped breakwaters it is generally known that oblique waves have no or little effect on the stability compared to the normal incidence case. However, for low crested breakwater where significant overtopping is allowed the overtopping causes larger forces on the back sections (both trunk and roundheads). As indicated by e.g. Juhl (1994) the overtopping can be larger for slightly (10°) oblique waves. Juhl stated that this effect is especially pronounced for low breakwaters. This lead to the feeling, that LCS's (especially the heads) might be more vulnerable under slightly oblique waves.

The confusion about the influence of short-crested waves on the stability of non-overtopped breakwaters, and the fact that no 3D experiments with oblique waves and low-crested breakwaters exist, indicate that experiments on LCS's subject to short-crested and oblique waves are needed.

Currents

Currents can directly affect the stability and indirectly e.g. by changing the shape of waves and/or by triggering wave breaking. Currents can be created e.g. by tide or storm surges, but localized currents generated by the presence of the structures may even be more important for LCS's. Low LCS's allows for significant overtopping. If the LCS's at the same time are long, then very large rip currents may be generated in the gaps. This will directly effect the stability especially by the roundheads, but also problems with scour may occur, see more hereabout in Chapter 9. The direct effects of currents on LCS armour layer stability are most likely important. However, the effects are still not fully clarified and this thesis provides no further investigations about current effects.

Surge and tidal water level variations

Water levels are very important especially for the stability of LCS's. First of all the water levels determines the freeboard, which is one of the dominating parameters for the stability of LCS's. Secondly the waves and water levels are strongly correlated, as many LCS's are built in shallow water.

4 Ecological aspects

Building LCS's along a natural shore will result in changes to the environment and/or associated living resources and trophic structure. The changes are mainly due to loss of natural soft-bottom habitats and associated assemblages with replacement of hard-bottom (rock) habitats. LCS's increases the diversity (i.e. qualitative increases the species richness), but soft bottom shallow water assemblages are highly disturbed due to the action of waves and currents modifying sediment location and composition. Types of species and ecosystem functioning may change considerably, with long-lasting possibly irreversible losses, making restoration extremely difficult and expensive. The LCS design should therefore meet specific management goals related to the environment, by mitigating impacts on the existing habitats or, when desirable, enhancing specific natural resources in a sustainable manner. Enhancing fish recruitment or promoting diverse assemblages for eco-tourism may be desirable because they are harvestable (e.g. mussels, crabs, oysters, limpets) or are enjoyable when bird watching or snorkelling.

Knowledge and forecast models about ecological consequences introduced by LCS's are in general very limited, but within DELOS some tools for identification and mitigation of ecological implications were developed. Broad qualitative forecasts of the kinds of species and the sequences of change on or around a defence structure can today be made with some confidence. However, quantitative predictions of the effects on individual species and assemblages at any particular location are still (too) difficult.

4.1 Natural rocky habitat versus artificial rocky habitat

A natural rocky habitat and the habitat in a LCS-scheme share some biological features, but are different in other ways, see Figure 4.1.

Figure 4.1. Photos of rocky habitats. Left: A natural habitat (Tjärnö, Sweden); right: An artificial habitat (Elmer, UK). Photos by Morten Kramer.

According to Moschella *et al.* (2005) LCS's share common physical and biological features with natural rocky shores in being:

- Subject to same natural processes (colonisation and succession, disturbance, recruitment fluctuations) leading to spatial and temporal variation
- Influenced by same physical factors (wave exposure, vertical stress gradients, salinity, etc.) and biological interactions (competition, grazing, predation)
- Same major functional groups (ephemeral green algae, canopy forming fucoids, filter feeding mussels & barnacles, grazing molluscs)

However, LCS's and other structures in general differ from natural LCS's and other structures in having:

- Limited extent
- Low habitat diversity
- High rate of disturbance

4.2 Ecology at existing LCS's

A considerable part of the DELOS project was devoted to the study of the ecology at existing structures, and to provide information about expected ecological impacts of new structures, both at the structures and in the near and far fields. Some brief ecological details about 150 specific European sites with LCS's are available in the DELOS inventory on LCS's, see Kramer (2001 & 2002). Detailed studies on abundance and composition of colonising epibiota were made on several shore-parallel LCS's located in Spain, Italy, Denmark and UK, see Moschella *et al.* (2005). In the following a few ecological findings from the Danish DELOS study sites are summarized. The text is based on the Danish study site report by Kramer and Dinesen (2004) included in Appendix B.

On microtidal shores in Denmark the influence of water depth on the composition of subtidal epibiotic assemblages was investigated, by recording the total number of species at half meter depth intervals from the water surface to a depth of 2 m on 5 replicate vertical transects on LCS's at Skagen, Lønstrup and Hirthshals. Although no formal comparison was possible due to the lack of nearby natural rocky shores, epibiotic communities on LCS's in Denmark appeared to have low diversity.

The total number of epibiotic species at 2 m depth was more than three times higher than at 0.5 m depth at both Skagen and Hirthshals. The increase in diversity with depth was probably related to less disturbed conditions in comparison to the wave-swept zone. On micro- and mesotidal shores, where epibiota occur mainly subtidally, diversity is higher on parts of LCS's located at greater depths.

Similar investigations and findings were performed at the other DELOS study sites in Spain, Italy and UK leading to the design features given subsequent.

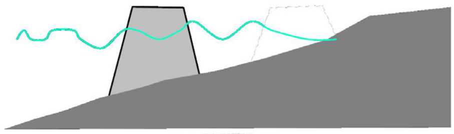
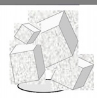
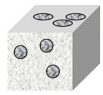
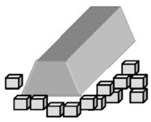
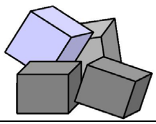
4.3 Design features affecting the ecology

Table 4.1 and 4.2 shows examples of means to increase the number of species at and near the structure by changing the position of the structure to deeper water and by inclusion of a range of surface roughness as well as providing scour protection.

Table 4.1. Examples to illustrate how target effects on epibiota could be obtained via intervention/modification of LCS design. For example, engineering intervention can influence diversity and abundance of epibiotic species by increasing surface and habitat complexity or locating the structures lower on the shore. These modifications can also lead to indirect effects, such as promotion of natural resources (shellfish and fish) and recreational activities. Moschella *et al.* (2005).

Management goal	Minimize disturbance to epibiota	Maximise habitat diversity	Maximise species diversity
Engineering intervention	- Flange around LCS - Minimal maintenance - Increasing stability of structure	- Making building blocks with rougher surface and pits - Building artificial rock pools	- Building LCS lower on the shore - Enhancing habitat diversity
Target effect	- Reduced disturbance - Colonisation by later successional species (e.g. limpets, mussels)	- Increased habitat and surface complexity - Increased species diversity	- Increased vertical extension of LCS below mid tidal level - Increased diversity, particularly for lower intertidal / subtidal species (e.g. kelp)
Other effects (indirect)	- Reduction in nuisance green algae - Enhancement of living resources (increase in shellfish)	- Promotion of recreational activities (e.g. swimming, rock pooling)	- Enhancement of living resources (shellfish, fish and crustaceans) - Promotion of recreational activities (sport fishing)

Table 4.2. Design features affecting habitat and disturbances.

Position on the shore	
Boulder arrangement, pools (< 5 m)	
Increase surface complexity (< 5 cm)	
Increase surface roughness (< 1 cm)	
Reduce scour (built wide stable toe berms)	
Increase stability (use larger armour blocks)	

Martin *et al.* (2005) states that from an ecological view the sheltering effects of LCS's in general should be minimized by keeping the modifications of both the onshore wave transport and water flow to the minimum necessary. Possible interventions are: maximise the overtopping and the porosity of structure; maximise the gap size and their frequency within each LCS; minimise the structure length and number; avoid beach nourishment (specially if planned to be carried out in successive periods); and minimise the enclosure of the protected zone (avoiding lateral groynes if at all possible). In conclusion, the effects of LCS should always be minimised, the number of LCS should be reduced to the minimum necessary to protect the coast, avoiding large-scale effects of habitat loss, fragmentation and community changes. Table 4.3 gives a summary of possible effects in relation to specific physical factors.

Table 4.3. Summary of critical design features affecting the type and magnitude of effects of LCS's and other hard defense structures on coastal environments and associated biota. Airoldi *et al.* (2005).

Factor	Predicted effects
Amount of structures	Proliferation of LCS's can result in broad-scale alteration of the whole coastline and large-scale, long-term effects
Location	Geographical context and predominant habitat characteristics are major determinants of the regional species pool
Spatial arrangement	Distance from natural reefs and other artificial structures influences dispersal of species including non-indigenous species
Height/size/porosity of structures	Permeability influences hydrodynamic conditions and sediment characteristics around the structures, as well as the type of epibiota that grows at the landward side
Project lifetime and structural integrity	Frequent and severe disturbances, as those occurring from block overturning and maintenance, keeps assemblages to an early stage of succession and favours the development of opportunistic species
Construction material/ habitat complexity	Physico-chemical attributes may affect the local and regional distribution of epibiota

Severe human disturbance (i.e. from harvesting or trampling) negatively affect benthic assemblages, and maintains species abundance and diversity low. Human access and use of the LCS's should therefore be regulated.

4.4 Types of materials

The physical and chemical attributes of materials used to build LCS's will affect the development of the epibiota, see Burcharth and Lamberti (2006). In particular, if the LCS are built with materials that are not typical of the area (e.g. limestone in an area of granite bedrock or concrete blocks) this may affect the local distribution of species, providing suitable substrata for species that would normally be rare or absent in the area, including invasive species. For example certain types of smooth geotextiles may be colonised only be ephemeral algae which can represent a nuisance for the local community. Therefore the same or similar stone materials typical of the area should be used. Carbonate rocks used for construction of LCS's are softer and are more easily weathered and bioeroded, leading to a more complex topography (crevices, small pits) which enhance colonisation and growth by algae and marine invertebrates.

A rough surface with crevices and small pits provides marine organisms a better protection from wave action, desiccation and insulation stresses and refuges from predators and grazers. As a result, a higher number of species can settle and survive. In general, the rougher is the surface the greater is diversity and abundance of epibiotic species. Natural rocks are generally characterised by these complex features, especially those that are more easily weathered, such as carbonate rock (e.g. limestone).

Colonisation of epibiota on concrete can be very different depending on the surface roughness. Very smooth concrete blocks are poorly colonised with settlement of very few species. Results from DELOS showed that when the concrete is rough there are no differences in the epibiota between this material and the natural rock. If concrete is used, a rougher surface texture should be preferred. Cast concrete can also integrate features such as small rock pools or holes that can promote colonisation by epibiotic species, crustaceans and fish.

Geotextiles does not offer a suitable substratum for colonisation by marine life unless it is very textured. Results from DELOS showed that organisms such as barnacles and mussels cannot colonise smooth surfaces, and ephemeral green algae are generally the only species present.

4.5 Structural stability and scouring at LCS's

The effect of maintenance of LCS's on epibiotic assemblages was examined on the DELOS Cesenatico defence scheme in Italy, see Moschella *et al.* (2005). The epibiota was compared between two LCS's that had been just repaired and two other LCS's that had not been maintained for at least three years. Qualitative observations were also made on various LCS's in the UK, Italy and Spain to assess the effect of scouring by sediment, variation in the sediment level and siltation on the epibiotic assemblages. Disturbance by sand scouring was preliminarily investigated at the Elmer field site in UK by recording the abundance of organisms on the surface of LCS blocks at increasing heights from the sediment level.

The effect of regular maintenance of LCS's through the addition of new building material to compensate for storm damage or sinking had dramatic effects on epibiotic communities on LCS's by the Adriatic coast. At Cesenatico, epibiota on structures that had just been repaired was much less diverse than on structures that had not been maintained for three or more years. In particular, the abundance of filter feeders, mainly mussels, was significantly reduced on recently maintained LCS's, whilst filamentous green algae increased. On heavily maintained LCSs, epibiotic assemblages seemed to be reset to early stages of colonisation.

Periodic maintenance should be minimized as stable structures, requiring minimal maintenance (including beach nourishment), allows development of mature assemblages. Airoldi *et al.* (2005) describes that from an environmental point of view the project lifetime and required maintenance is one of the most crucial factors affecting the composition, abundance and distribution of species that colonise the structures themselves. Frequent and severe disturbance effectively maintains assemblages at an early stage of succession, with few species compared to those on structures which have not been maintained, and favours the development of green ephemeral algae, with consequent negative effects on the quality of the beach. For any new structure introduced into the marine environment, it will take time for mature biological communities to develop. Thus, to promote mature assemblages, coastal defence structures need to be stable and built in such a way that maintenance will be minimal. Unless defence structures meet these criteria, there is little point in introducing additional features to meet specific secondary end-points (for example, enhancing habitat complexity to promote diverse assemblages for ecotourism), as attempts to repair the structure will result in considerable degradation of developing assemblages.

Maintenance costs could be determined with reference to the expected damage during LCS lifetime as predicted by stability formulae, though the total costs will increase due to the higher mobilization costs of the equipment for a small volume of rock to be placed, see Burcharth and Lamberti (2006). Moreover it should be kept in mind that most detached LCS's are without permanent groynes and therefore the possible access causeway for trucks and cranes needs to be reconstructed and demolished again to reach the offshore structure; moreover, when the LCS is submerged any new addition of material may require a new emerged mound to be later dismantled. Thus maintenance works for LCS are relatively expensive and with a strong disturbance to the local ecology and recreational activities and should therefore be reduced to a minimum or avoided with a more conservative and careful design.

5 Armour layer stability

Within the DELOS project the author performed 3D and 2D model tests at the Hydraulics and Coastal Engineering Laboratory, Aalborg University (AAU), Denmark. The tests are referred to as *AAU 2002* and *AAU 2005* respectively. The task for the AAU stability tests on LCS's (mainly roundhead but also trunk) was to supplement existing tests in order to identify the influence on rubble stone stability of: Obliquity of short crested waves, wave height and steepness, crest width, and freeboard. Further design formulae were extracted from the experimental data in order to provide recommendations for design as given in Kramer and Burcharth (2003) and Burcharth *et al.* (2006). The present chapter gives an overview of existing tests and formulae compared to the new tests and formulae, whereas much further detail is given in Appendix C to G.

As most LCS's are relatively low (H_c < 4 m) and are built close to the beach in fairly shallow waters the investigation has focussed on matching the tools to such conditions. Two new equations Eq (5.1) and a rule of thumb Eq (5.2) has been derived to meet the design conditions, see Table 5.1. The table is based on knowledge about existing structures (see Table 5.7), the behaviour of Eq (5.1) and the rule of thumb.

Table 5.1. Design conditions for armour layer stability with respect to waves and water levels.

<table>
<tr><th>Structure height</th><th>Freeboard at MWL</th><th>Design freeboard and water depth</th><th>Design waves</th><th>Design tool</th></tr>
<tr><td rowspan="2">H_c < ~4m</td><td>Slightly emerging to slightly submerging</td><td>Worst condition is for $R_c/H_c \cong -0.3$ if obtainable. Typically the highest design water depth is the worst condition.</td><td>Depth limited</td><td>Rule of thumb</td></tr>
<tr><td>Very submerging ($R_c/H_c < -0.4$)</td><td>Worst condition is for $R_c/H_c \cong -0.3$ if obtainable. Typically a frequently occurring low water level or even the lowest design water depth is the worst condition.</td><td>Depth limited</td><td>Rule of thumb or if very submerging Eq (5.1)</td></tr>
<tr><td rowspan="3">H_c > ~4m</td><td>Very emerging structures</td><td colspan="3">Not a low crested structure</td></tr>
<tr><td>Slightly emerging to slightly submerging</td><td>Worst condition is usually for the highest design water level.</td><td>The design waves may not be fully depth limited ($H_s/h < 0.6$)</td><td>Eq (5.1)</td></tr>
<tr><td>Very submerging ($R_c/H_c < -0.4$)</td><td colspan="3">Structures does not exist. However Eq (5.1) may still be used for design.</td></tr>
</table>

The following chapter describes the background for the new formulae. Chapters 5.2 and 5.3 describe the model tests at AAU and in Chapter 5.4 the new datasets are compared to existing datasets. Based on the datasets new formulae are given in Chapters 5.5 and 5.6. In Chapter 5.7 the new formulae are compared to existing design formulae, and in Chapter 5.8 the new formulae are validated by prototype experience. As damage is only given according to *initiation of damage* Chapter 5.9 describes the residual stability. An example of the use of the equations is finally given in Chapter 5.10.

5.1 Earlier trunk and roundhead stability tests

Several researchers have investigated trunk armour layer stability of LCS's; see e.g. Powell and Allsop (1985), Givler and Sorensen (1986), Ahrens (1987), Van der Meer (1988), and Loveless and Debski (1997). However, the most extensive work was performed by Vidal *et al.* (1992), Burger (1995), and Kramer and Burcharth (2003), which is described in more detail in the following.

Vidal *et al.* (1992) performed laboratory experiments at NRC (the Hydraulics Laboratory of the National Research Council, Canada) on a complete 3D structure to investigate trunk and roundhead damage. The experiments and elaboration of results are given in Vidal *et al.* (1992, 1995 and 2000). The cross section had slopes 1:1.5 on both seaward and landward sides and a crest width of $6D_{n50}$, see Figure 5.1. The waves were non-depth limited and perpendicular to the trunk. Vidal showed that the trunk crest was the least stable part of the structure in case of submerged structures, and that the leeward part of the head was the least stable part under emergent conditions, see Figure 5.2.

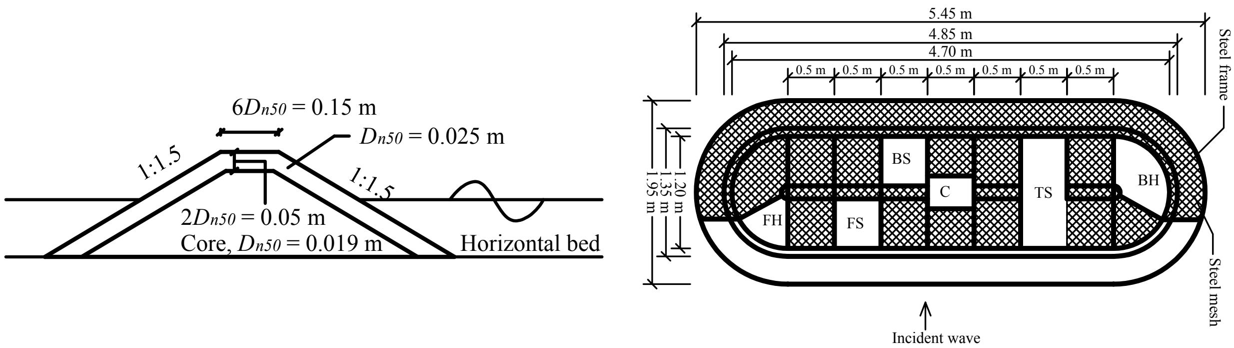


Figure 5.1. Plan view and cross section of model by Vidal et al. (1992).

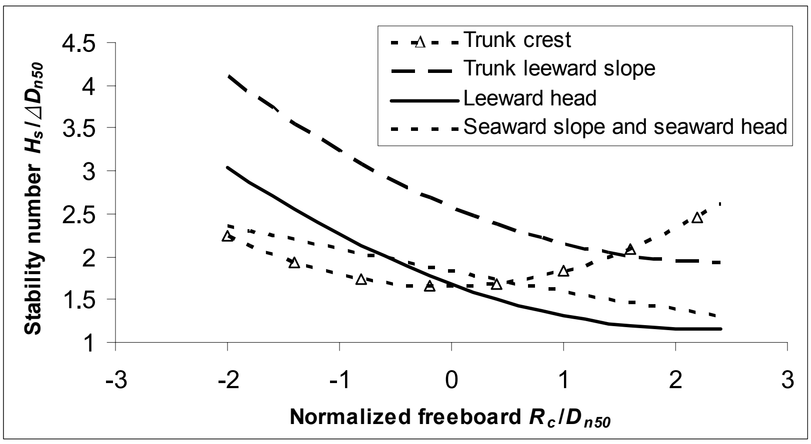


Figure 5.2. Design diagram for LCS armour stability, initiation of damage. Vidal et al. (1992, 1995). Non-depth limited waves.

Vidal *et al.* (1992) divided the structure into several sections in order to study the distribution of the damage. It should be noted that the definition of crest in these tests contained the upper parts of the two slopes. A steel frame was covering the surface of the structure along the sections, and a steel mesh was covering the parts where damage was not measured. Damage interactions among the sections were thereby not possible, e.g. damage to the crest section could not influence damage to the seaward slope section and visa versa. Further the steel frame restricted stones from movements along the boundaries within the sections. These effects most probably stabilized the stones making the sections in the experiments more stable than what would be the case for real structures. Vidal *et al.* (1992) also studied the response of a complete trunk section without steel mesh covering. The results are implemented in Figure 5.4.

Burger (1995) performed new laboratory experiments at Delft Hydraulics on trunk stability and re-analysed the existing tests reported by Vidal *et al.* (1992) and Van der Meer (1988). The cross sections of Van der Meer and Burger had slope 1:2 at the seaward side and slope 1:1.5 at the landward side, see Figure 5.3. The crest width, armour layer thickness and core material characteristics in the tests by Burger (1995) is not reported in any existing publication. The waves were non-dept limited and perpendicular to the trunk. The analysis is described in detail in Burger (1995) and is summarized in Van der Meer *et al.* (1996). The trunk was divided in seaward slope, crest and leeward slope. Related to initiation of damage stage the stability was reported both for each sector and for the total trunk sector, see Figure 5.4. From the figure it is seen that the crest is the least stable part of the trunk under submerged and slightly emergent conditions. For more emergent conditions the seaward slope is the least stable part.

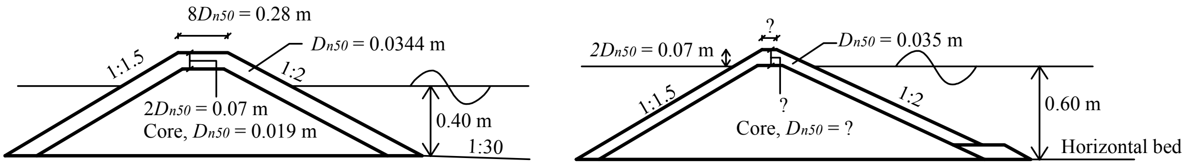


Figure 5.3. Cross section of Van der Meer 1988 tests (left) and Burger 1995 tests (right).

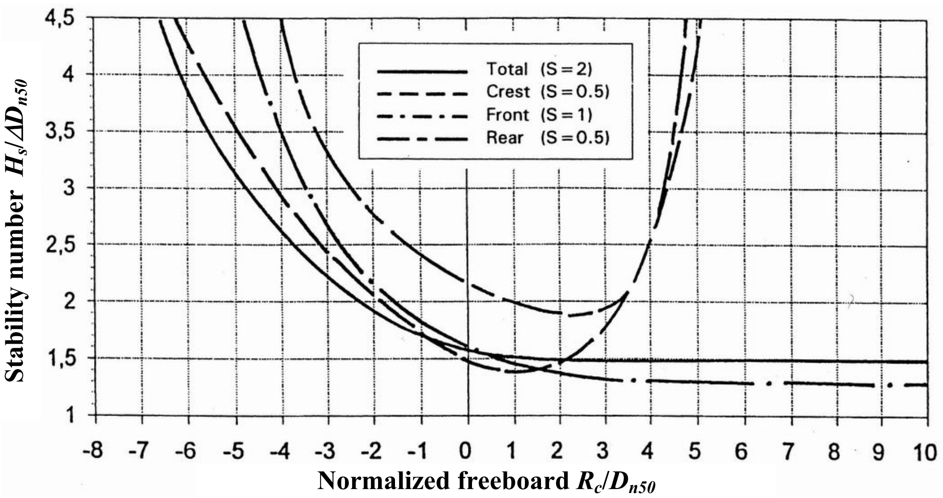


Figure 5.4. Design diagram for trunk armour stability for initiation of damage, based on tests by Van der Meer (1988) and Vidal *et al.* (1992). Burger (1995). Non-depth limited waves.

Much further detail about the tests and design diagrams are given in Appendix D10, D11, F and G.

5.2 AAU 2002 tests: 3-D model tests with shallow water waves

In the following a brief description of the AAU 2002 tests is given, for details please see Appendix D. The chosen set-up was based on a survey of the geometry of 1248 existing low crested breakwaters in the EU as described in Chapter 2 and Appendix A. Typical ranges of structural geometries were identified and scaled by 1:20 leading to appropriate sizes of the structures with respect to the size of the wave basin. The largest possible armour stone sizes were chosen based on existing knowledge about stability of LCS's and the obtainable wave conditions in the basin. In this way damage to the structure was likely for the highest waves.

Sufficiently large Reynolds numbers were ensured to avoid problems with viscous scale effects (the Reynolds numbers were about 3 to $5 \cdot 10^4$ in the tests leading to initiation of damage).

69 tests were performed with irregular 3D waves generated using a Jonswap spectra with peak enhancement factor 3.3 and a cosine power spreading function with spreading parameter $S_\theta = 50$, see Mitsuyasu *et al.* (1975). The wave height was increased in steps until severe damage occurred. Two wave steepnesses of s_{0p} = 0.020 and 0.035 and angle of incidences in the range -30° to +20° were generated (0° = normal incidence). The water depth in front of the wave paddle was varied from 33 cm to 48 cm giving water depths at the structure between 0.25 m and 0.40 m. Two different crest widths were tested at different water levels giving freeboards between -0.1 m and +0.05 m, see Figure 5.5. Negative freeboards represent submerged structures. The length of the structure was 5 m. A circular roundhead with crest radius equal to half the trunk crest width was chosen. The structure was located at a plateau 8cm above the seabed at the paddles. The plateau was built by flagstones, and the foreshore slope was poured in concrete.

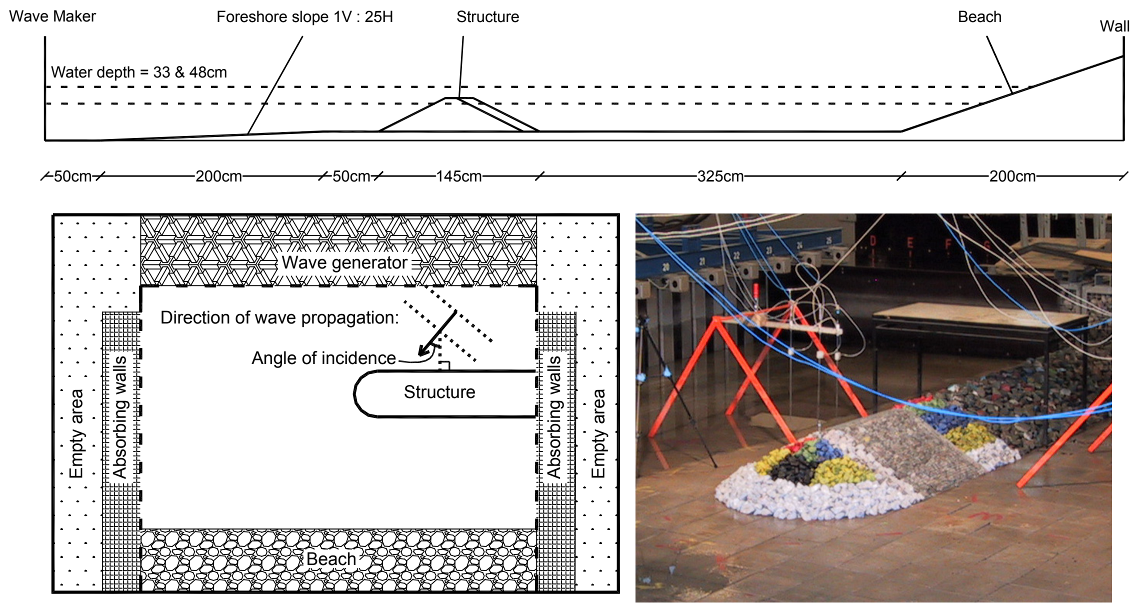
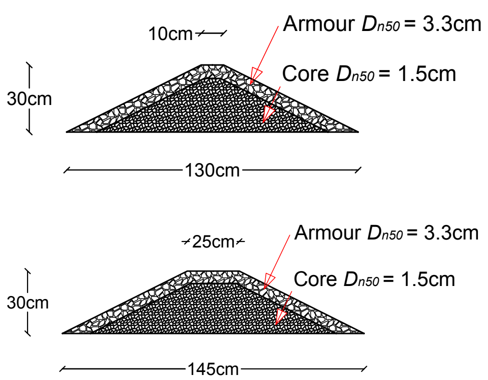


Figure 5.5. Wave basin layout and bottom topography in 3D tests at Aalborg University 2002.



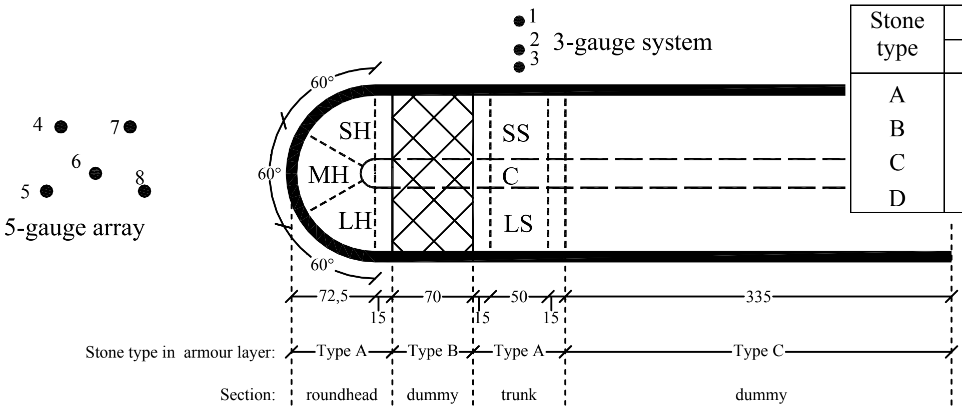
Parameter	Value
Crest width B	$3D_{n50}$ and $8D_{n50}$
Crest height H_c	0.30 m
Structure slope	1V : 2H
Freeboards R_c	-0.10, -0.05, 0.0 and 0.05 m
Armour stone size	D_{n50} = 0.033 m
Core stone size	D_{n50} = 0.015 m
Armour layer thickness: 0.066 m ($2D_{n50}$)	

Figure 5.6. Cross-section geometry for 3D tests at Aalborg University 2002.

LCS construction and materials

The trunk and the roundhead were constructed by carefully selected quarry stones with mass density 2.65 t/m^3. Three types of armour stones were used. Carefully selected stones (Type A) were used in the test sections where damage was measured, see Figure 5.7. For the dummy section between the trunk and roundhead test section a net with large masks (2x2cm) was covering the surface to avoid damage in that area. This made rebuilding easier and gave less strict specifications for the armour material (Type B). For the dummy section between the side-wall (to the right on Figure 5.7) and the trunk test section, larger stones (Type C) were used to avoid damage. Type A stones were used in 15 cm ($5 \cdot D_{n50}$) strips on each side of the test sections to ensure correct boundary conditions. More wide graded stones (Type D) were used as core material. The porosity n for armour Type A and core Type D was $n_{(\text{Type A})} = 0.44$ and $n_{(\text{Type D})} = 0.43$.

The roundhead was split in three sections of 60° each. The three sections were called: Seaward Head (SH), Middle Head (MH) and Leeward Head (LH). The trunk was split in three parts called: Seaward Slope (SS), Crest (C), and Leeward Slope (LS). The damage within each section could thereby be measured from digital photos. The stones in each section were painted in different colours to identify and quantify damage, see Figure 5.8.



Stone type	Stone sizes	
	D_{n50} [cm]	D_{n85}/D_{n15}
A	3.25	1.20
B	3.07	1.28
C	4.74	1.21
D	1.44	1.64

SH	Seaward Head
MH	Middle Head
LH	Leeward Head
SS	Seaward Slope
C	Crest
LS	Leeward Slope

Figure 5.7. Stone types in structural sections. Measures in cm.

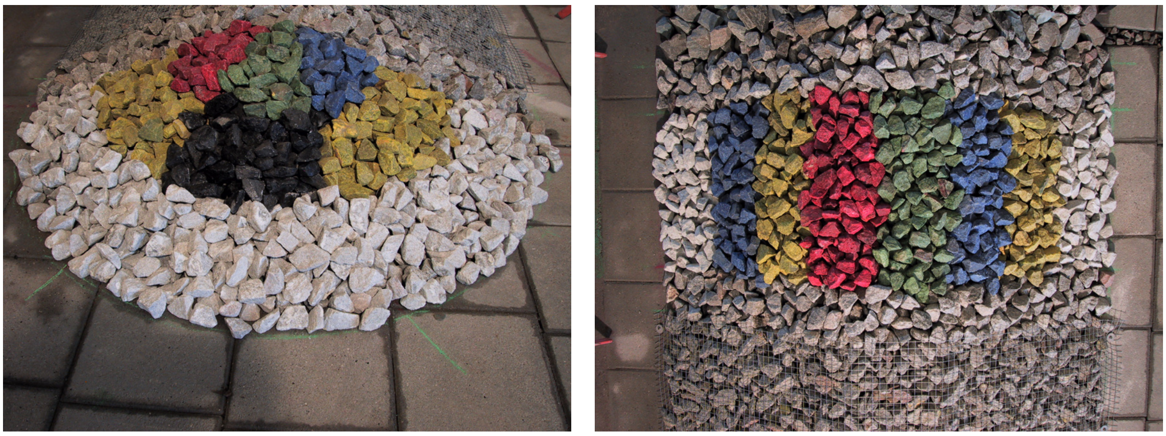


Figure 5.8. Colouring of stones in roundhead (left) and trunk (right). Waves are coming in from the left.

Measurements

Waves were recorded by an array of five wave gauges of the resistance type to be used in estimating incoming and reflected wave spectra, see Figure 5.7. At the position of the array almost 1.5 metres from the roundhead the influence of the roundhead (reflection and diffraction) on the incoming waves is believed to be negligible. However, the trunk reflects some wave energy which is re-reflected by the paddles. Therefore the waves in front of the trunk might in reality be slightly higher with more wave breaking than at the array. Measurements from the 3-gauge system and visual observations were performed to quantify this effect. Recorded waves were analyzed with the software Wavelab©, developed by the laboratory at AAU (http://hydrosoft.civil.auc.dk/).

Digital video and digital photos were taken to visualize and quantify the damage progression.

Results

The target length of each test was 1000 waves. A test block was defined by fixed water level, wave direction, and wave steepness, see Table 5.2. In each test block the significant wave height H_s was increased in steps until severe damage was observed. It was attempted to get four tests in each block. However, this was not possible in all blocks due to the progress of the damage. Target conditions were therefore continuously adjusted according to target damage during a tests block. After each block the breakwater was rebuilt.

Table 5.2. Test conditions in the 3-D tests at AAU 2002. β is the main wave direction.

Narrow crest (width = 0.1m)			
Test block	β [°]	Freeboard [m]	Wave steepness s_{0p}
1	0	0.05	0.020
2	0	0.05	0.035
3	0	0.00	0.020
4	0	0.00	0.035
5	0	-0.05	0.020
6	0	-0.05	0.035
7	0	-0.10	0.020
8	0	-0.10	0.035

Wide crest (width = 0.25m)			
Test block	β [°]	Freeboard [m]	Wave steepness s_{0p}
9	0	0.05	0.020
10	-20	0.05	0.020
11	-10	0.05	0.020
12	10	0.05	0.020
13	20	0.05	0.020
14	-30	0.05	0.020
15	0	0.00	0.020
16	0	-0.05	0.020
17	0	-0.10	0.020

A detailed report about the tests is available in the deliverables for the DELOS project, see Kramer *et al.* (2003). An overview of the experimental layout can be found in Kramer *et al.* (2004). Based on Figure 5.9 Kramer and Burcharth (2003 and 2005) gave some recommendations for design. They are repeated in the following.

For the trunk the crest is the least stable section and the leeward slope the most stable part, see Figure 5.9. For the roundhead the leeward head is the least stable part, and the stability of the middle head and seaward head is approximately the same. The trunk crest is the least stable part under submerged conditions, and for emergent conditions the leeward head is the least stable.

Wave direction. All parts of the trunk are slightly more stable under oblique wave attack than under normal incidence wave attack. The stability of the roundhead sections in case of oblique waves with $\beta < 0°$ (i.e. a large part of the head exposed to direct wave attack) is the same as for normal incidence waves. The stability of the leeward and middle part of the roundhead in case of oblique waves with $\beta > 0°$ (i.e. when a large part of the head is in lee of direct wave attack) is the same as for normal incidence waves, but the area of damage shifts towards the middle part of the head. During the experiments it was experienced that wave breaking tends to focus at the roundhead forming a jet of water slamming down on the top part of leeward head. This effect shifted towards the middle head in case of oblique waves causing the middle head more prone to damage.

Wave steepness. The investigation showed that the test-data for $s_{0p} = 0.020$ and $s_{0p} = 0.035$ were fairly close. However, the series with $s_{0p} = 0.020$ (long waves) tend to give slightly more damage than series with $s_{0p} = 0.035$ (short waves) meaning the structure is more stable for $s_{0p} = 0.035$.

Crest width. No significant difference in response could be identified for the tested crest widths indicating that for the tested range the influence of crest width was small. For much wider crests than tested the influence will be significant, giving a more stable structure.

Freeboard. The tests showed that the stability is highly influenced by the freeboard.

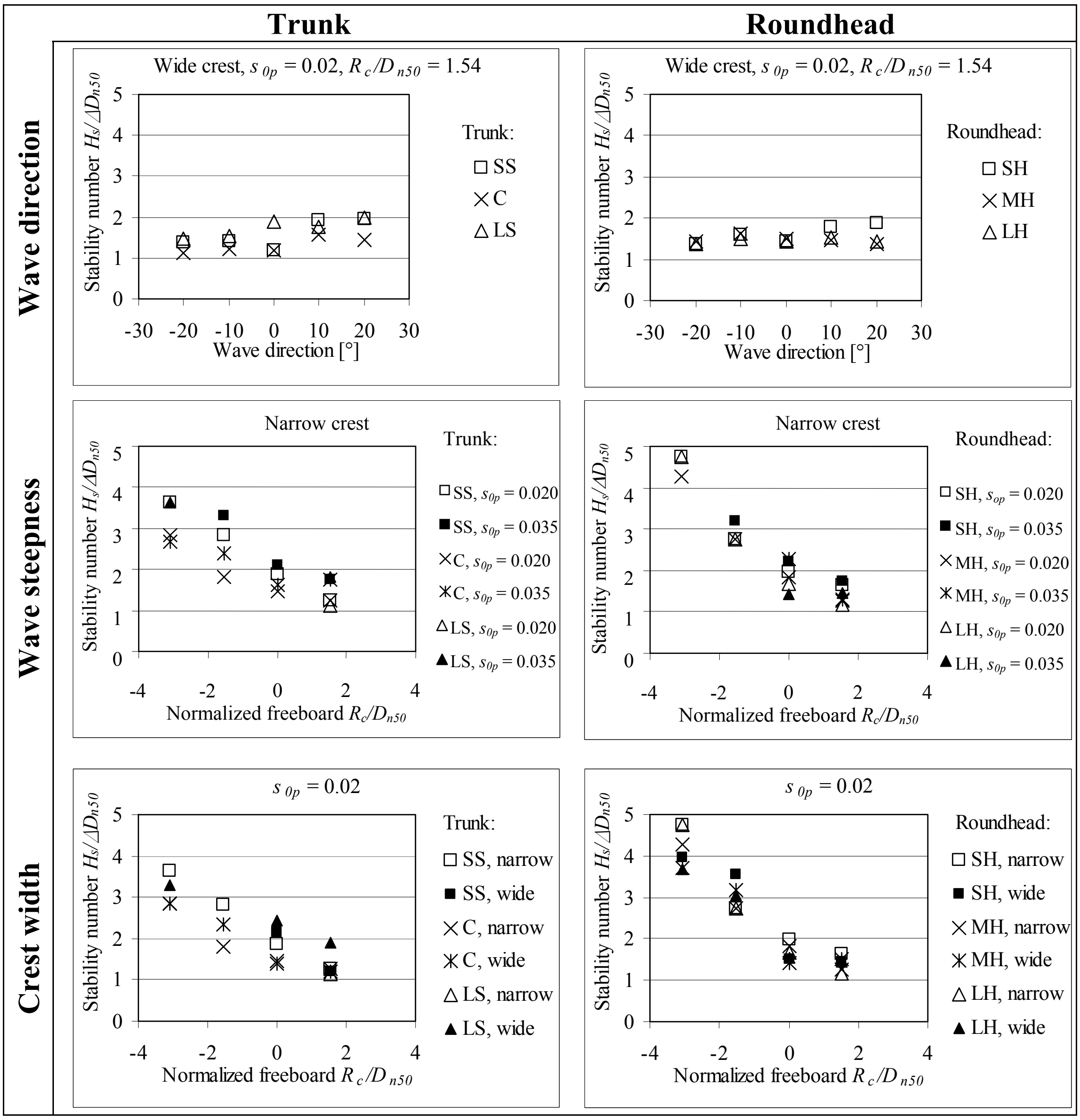
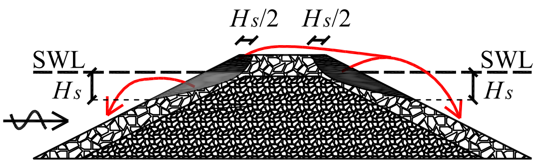
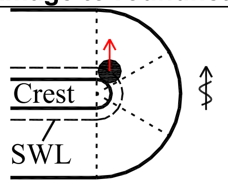
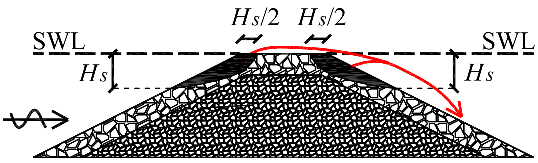
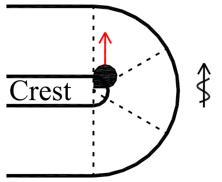
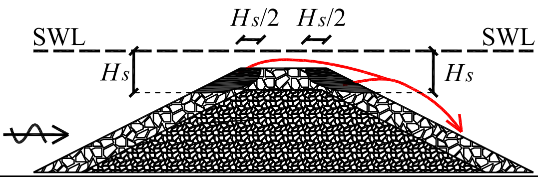
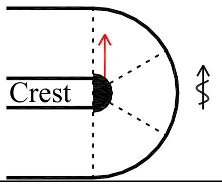


Figure 5.9. Results of 3-D experiments at Aalborg University 2002 for initiation of damage.
Legend trunk: SS: Seaward Slope, C: Crest, LS: Leeward Slope
Legend roundhead (60° sectors): SH: Seaward Head, MH: Middle Head, LH: Leeward Head

Kramer and Burcharth (2003) described the exposed areas of the breakwater as given in Table 5.3. The information in the table is important if there is a wish for optimization by using different stone sizes in the different parts of the armour layer.

Table 5.3. Sections prone to damage based on 3D tests at Aalborg University. Filled black areas indicate exposed stones, and red arrows the directions of armour unit displacements in case of damage.

Freeboard	Damage to trunk	Damage to roundhead
$R_c > 0$ Slightly emergent crest		
$R_c = 0$		
$R_c < 0$ submerged crest		

In the AAU 2002 tests the trunk and the roundhead were divided in different sections and damage was measured within each section, see Figure 5.10. Low narrow crested breakwaters built in shallow water are only a few stone-sizes high and wide. One stone removed from the edge of the crest will cause a large hole in the cross-section. When one section reached the initiation of damage stage it was therefore chosen to define the whole structure to be in this stage. In Figure 5.10 a line representing the lower limit of the test results is given. This line represents the least stable part of the structure. The function for the line is given below by Eq (5.1).

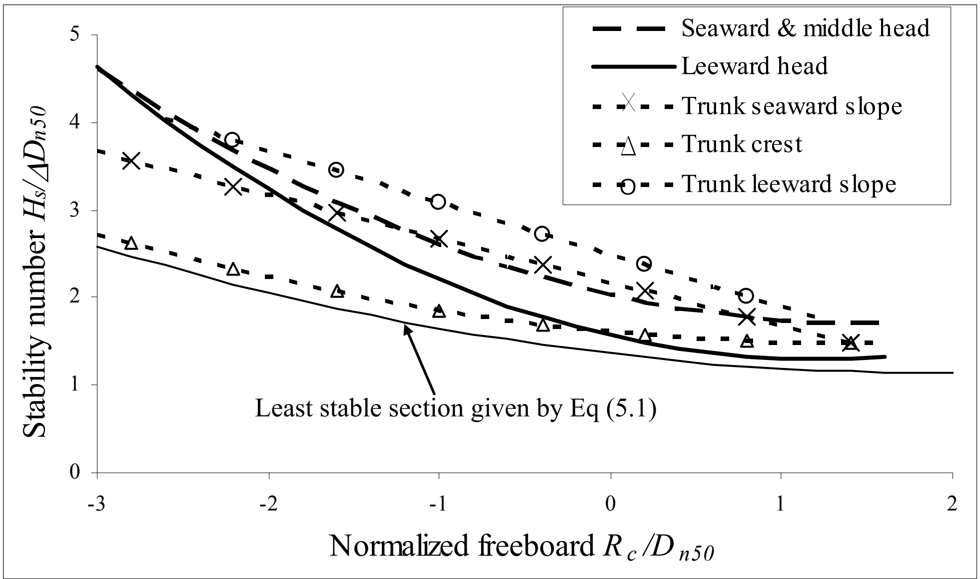


Figure 5.10. Design diagram for LCS armour stability, initiation of damage. Kramer *et al.* (2003). Shallow water waves.

The applied ranges of H_s/h corresponding to damage initiation are given in Table 5.4. It is seen that the observed stage of damage initiation corresponds for the tested structure to the range H_s/h = 0.25 to 0.40 for the trunk and H_s/h = 0.25 to 0.60 for the roundhead. This shows that in most cases the damage to the tested structure starts before H_s reaches the maximum depth limited heights, which, for the given foreshore slope and the applied wave steepnesses, would be approximately H_s/h = 0.5 to 0.6. Only when the crest is relatively deeply submerged $R_c/D_{n50} < -2$) the waves could grow to their depth limited maximum height without causing damage to the main armour. This is not a general rule but is a characteristic of the tested structure and structures with a low value of D_{n50}/h. In order to examine structure response for larger values of D_{n50}/h additional tests were performed at AAU as discussed in Chapter 5.3.

Table 5.4. Depth limitation of waves corresponding to initiation of damage in the 3D tests at Aalborg University. Tested ratios of H_s to water depth at the structure h are given.

Parameter	Water depth h (m)			
	0.40	0.35	0.30	0.25
R_c/D_{n50}	-3.03	-1.51	0	+1.51
H_s/h for trunk	0.35 – 0.40	0.30 – 0.35	0.25 – 0.30	0.25 – 0.35
H_s/h for roundhead	0.50 – 0.60	0.40 – 0.50	0.25 – 0.30	0.25 – 0.35

5.3 AAU 2005 tests: 2-D model tests with depth limited waves

A series of 2D model tests with the structure shown in Figure 5.11 and Figure 5.12 exposed to depth limited irregular waves were performed at the Hydraulics and Coastal Engineering Laboratory at Aalborg University in order to verify the simple design rule given by Eq (5.2). Details about the tests are given in Appendix C.

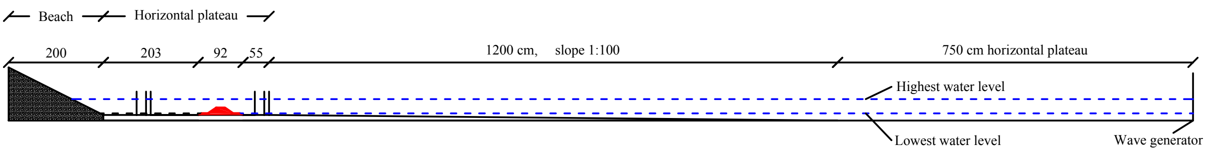


Figure 5.11. Layout in the 2D tests at Aalborg University. Structure cross-section is shown in red. Two wave gauge arrays with three gauges in each array are shown in front of and behind the structure. Measures in cm.

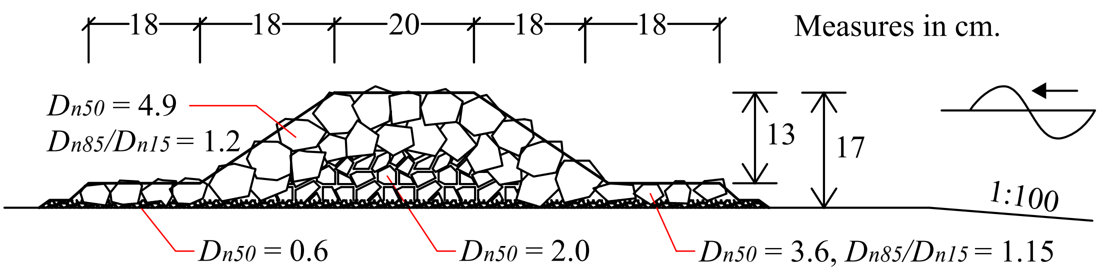


Figure 5.12. Cross section of breakwater tested in depth limited waves at Aalborg University.

The mass density of the rock and toe armour was 2.65 t/m^3. The total height of the structure was $H_c = 3.5 \cdot D_{n50} = 17$ cm, corresponding to $D_{n50} = 0.29 H_c$ as given by Eq (5.2). The water depth was increased in nine steps from 0.04 m to 0.34 m. Waves with spectral peak period of T_p = 1.8 sec were used together with the maximum possible wave heights at the structure corresponding to the actual water depth. Wave reflection compensation was used together with two triple wave gauge arrays. The incident waves at the structure were depth limited with significant wave heights in the range 0.43 to 0.51 times the water depth.

Damage to main armour and toe was recorded using digital photos taken before and after runs of approximately 1000 waves.

The tests showed that no displacements of main armour took place until water depth reached $h = 0.23$ m corresponding to $R_c = -0.35H_c$ and $R_c/D_{n50} = -1.2$, and in this case only one stone in the upper part of the front armour was displaced corresponding to initiation of damage. Increasing the water depth did not result in more damage. This result confirms the simple rule Eq (5.2) and shows that Eq (5.1) can be used for depth limited waves.

No active controlling of the set-up behind the trunk was performed. In Appendix C a detailed description of the waves and the set-up behind the trunk is given. In the tests with $R_c = -0.35H_c$ the wave steepness was $s_{0p} = 0.021$, which is typical for real wave conditions. Further the set-up in this test was $S_u/h = 0.027$, i.e. only slightly less than 3 % of the water depth, indicating that return-flows in this test was small.

5.4 Comparisons of datasets

The data sets described in Table 5.5 were compared in Kramer *et al.* (2003). Structure geometries, wave basin/flume layouts, stone characteristics and types of waves generated were different in all the datasets. However, when the differences are kept in mind, Kramer *et al.* (2003) concluded that all data sets are in reasonable agreement.

Figure 5.13 shows all the trunk and roundhead data available from the tests at the National Research Council Canada (NRC, 1992) described by Vidal *et al.* (1992), Delft Hydraulics (Delft, 1988 and 1995) described by Van der Meer *et al.* (1988 and 1995), and Aalborg University (AAU, 2002) described in Chapter 5.2. The sets of tests differ with respect to slope and waves (slope 1:1.5 and non-depth limited 2-D waves in NRC 1992 and Delft tests, slope 1:2 and shallow water short-crested waves in AAU 2002 tests). However, it happens that the effect of these differences compensates each other if only the stage of initiation of damage in some part of the structure is considered. This is indicated in Figure 5.14, by the lower envelope curve given by following stability formula Eq (5.1), which represents the initiation of damage in some part of the structure.

Table 5.5. Model characteristics for NRC, Delft and AAU tests.

Parameter	Test facility and year			
	NRC 1992	**Delft 1988 (trunk)**	**Delft 1995 (trunk)**	**AAU 2002**
Armour unit size D_{n50} [m]	0.025	0.034	0.035	0.033
Structure height H_c/D_{n50}	16.0	8.7, 11.6, 15.3	19.1	9.1
Crest width B/D_{n50}	6.0	8.0	?	3.0 and 7.6
Freeboard R_c/D_{n50}	-2, 0, 0.8, 1.6, 2.4	-2.9, 0, 3.6	2.0	-3.1, -1.5, 0, 1.5
Structure slope	1:1.5	1:2, leeward 1:1.5	1:2, leeward 1:1.5	1:2
Foreshore slope	Horizontal	1:30	Horizontal	1:20
Type of waves	2D irregular*)	2D irregular*)	2D irregular*)	3D irregular
Wave direction	Head on (0°)	Head on (0°)	Head on (0°)	-20° to +20°
Reference	Vidal *et al.* (1992)	Van der Meer (1988) and Burger (1995)	Burger (1995)	Kramer *et al.* (2003)

*) Non-depth limited waves

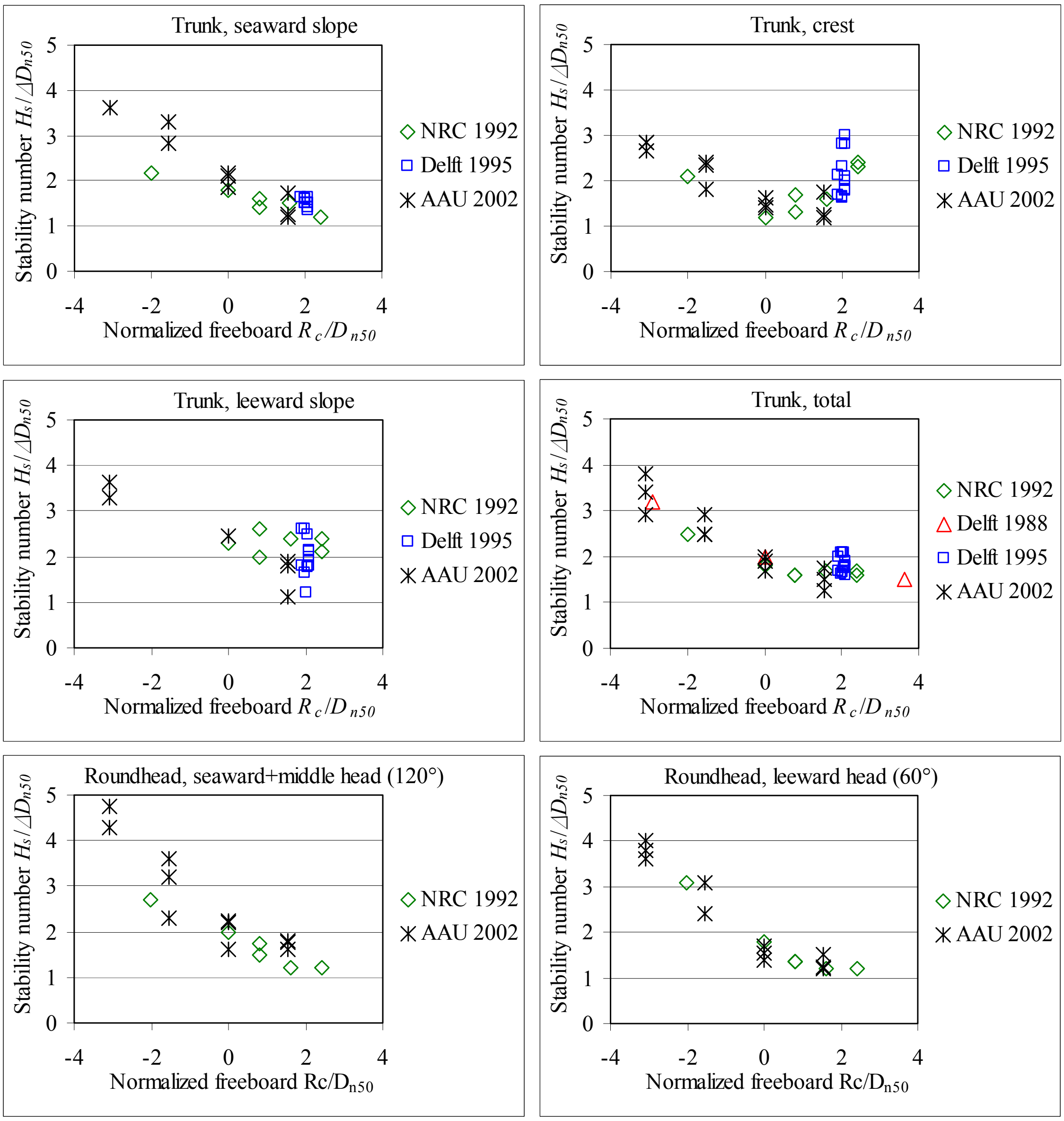


Figure 5.13. Test results for stability of low crested breakwaters corresponding to initiation of damage.

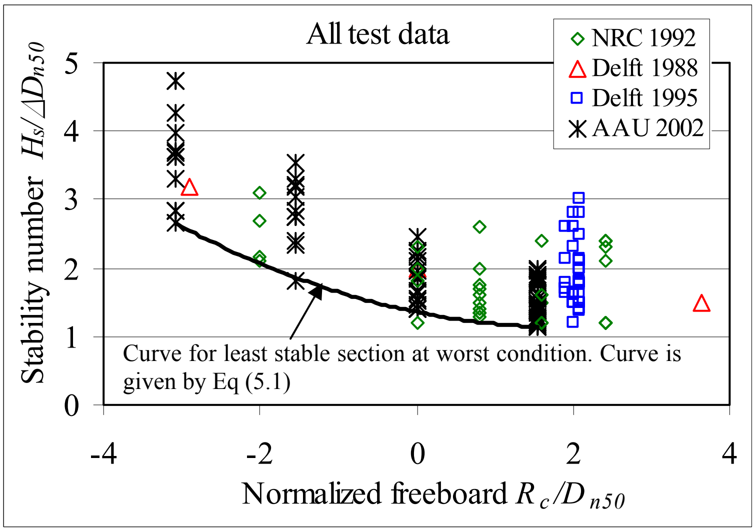


Figure 5.14. All available test results with LCS's corresponding to initiation of damage.

5.5 Design formula for required armour stone sizes in shallow water waves

When designing a low crested breakwater the highest significant wave heights must be calculated for different water depths caused by tide and storm surge. The corresponding necessary stone sizes for each of these water depths can then be found from the figures 5.2 to 5.18. In this way the "worst condition" will be the water depth giving the largest stone size. It is recommended to choose the stone size according to the lower line shown in Figure 5.10 and Figure 5.14 given by Eq (5.1).

$$\frac{H_s}{\Delta D_{n50}} = 0.06\left(\frac{R_c}{D_{n50}}\right)^2 - 0.23\frac{R_c}{D_{n50}} + 1.36 \qquad \text{, for } -3 \le R_c / D_{n50} < 2 \qquad \text{Eq (5.1)}$$

In Eq (5.1) R_c is the freeboard (negative if submerged), D_{n50} is the mean nominal diameter of the armour, and $\Delta = (\rho_a - \rho_w)/\ \rho_w$, where ρ_a and ρ_w are the densities of armour and water, respectively. H_s is the significant wave height at the breakwater location, and it can be estimated with a good accuracy by the method described in Chapter 6. An example of the use of Eq (5.1) is shown in Figure 5.15 and Figure 5.16. The validity of the formula is given below.

- **Freeboard.** The formula is only valid for relatively low freeboards given by the ranges in Eq (5.1). For more emergent structures design according to the upper limit of Eq (5.1) is most likely sufficient, or existing formulae for roundhead stability of non overtopped breakwaters can be used. The upper limit of Eq (5.1) is R_c / D_{n50} = 2 corresponding to a stability number of $H_s/\Delta D_{n50} = 1.14$, which in terms of stone size is $D_{n50} = H_s/1.14\Delta$.
- **Wave obliquity.** The formula is safe to apply also in case of oblique wave attack. The tests by Kramer *et al.* (2003) showed that wave directions in the range -20° to +20° leads to a slightly larger stability. However, the increase did not justify for a reduction in the necessary rock size within the tested range of obliquities.
- **Wave steepness.** The formula is tested for fairly long waves (s_{0p} = 0.02) and rather short waves (s_{0p} = 0.035). If extremely long waves are expected design by Eq (5.1) may underestimate the necessary stone size.
- **Stone-type.** The formula is only valid for armour material consisting of quarry rock. Tests were performed with rock density 2650 kg/m^3, but most likely the formula can be used in case of heavier or lighter types of rock. It is recommended to use grading $D_{85}/D_{15} < 2.5$ but no restrictions on rock shape seems necessary. The formula is developed for random stone placement, but it is on the safe side to use the formula for tightly packed structures.
- **Layers.** A two-layer moderately permeable rubble mound structure was tested. However, it is safe to use the formula for design of homogeneous structures. For multilayered or impermeable rubble mound structures caution should be taken if Eq (5.1) is used.
- **Slopes.** The breakwater should be built with slopes not steeper than 1:2. Breakwaters with less steep slopes are more stable and design by Eq (5.1) will therefore be safe.
- **Crest-width.** The formula is developed for narrow-crested breakwaters (crest widths less than approximately $10D_{n50}$). However, it is safe to use the diagrams or formulae for structures with wider crests as they will be more stable.
- **Trunk/roundhead differences.** The formula is based on the assumption that the same stone size and type will be used in all armouring parts of the breakwater. If there is a wish for optimizations by using different stone sizes in the different outer sections of the breakwater, design can be done according to the figures 5.2, 5.4, and 5.10. In this case important information about the location of the most exposed areas can be seen in Table 5.3.

5.6 Design formula for required armour stone sizes in depth limited waves

If the highest waves are depth limited then the significant wave height may be replaced by the approximation $H_s = 0.6 \cdot h$ (h is water depth). By inserting in Eq (5.1) regular rock density $\rho_r = 2.65$ t/m^3 corresponding to $\Delta = 1.6$, and $H_s = 0.6 \cdot h$ the curves in Figure 5.15 are obtained. Under breaking wave conditions, increasing water level increases wave load and the damage to the structure, until submergence reaches condition $R_c = -0.36 \cdot H_c$. Further water level increase will cause a dominant self protection of the structure by submergence. The R_c/H_c relation is used in Eq (5.1) to calculate the required D_{n50} and the rule of thumb is found:

$$D_{n50} = 0.29 \cdot H_c, \qquad \text{submergence can reach the critical criterion } R_c = -0.36 \cdot H_c \qquad \text{Eq (5.2)}$$

Please note the flat maxima for the curves in Figure 5.15 signifying that the rule of thumb is valid for a fairly large range of submergences even though the most critical freeboard is $R_c = -0.36 \cdot H_c$. According to Eq (5.2) the structure height will be no more than H_c = 3 to $4 \cdot D_{n50}$, which is very typical for existing LCS's. For other H_s/h values Figure 5.16 can be used in the design.

If the structure is emerged under design conditions the upper limit of Eq (5.1), corresponding to $D_{n50} = H_s / 1.14\Delta$, is most likely sufficient for design. By inserting ρ_a = 2.65 t/m^3 corresponding to $\Delta \approx 1.6$ and the approximation $H_s = 0.6 \cdot h$, the required stone size is

$$D_{n50} = 0.33 \cdot h., \qquad \text{structure cannot get submerged at the highest water level} \qquad \text{Eq (5.3)}$$

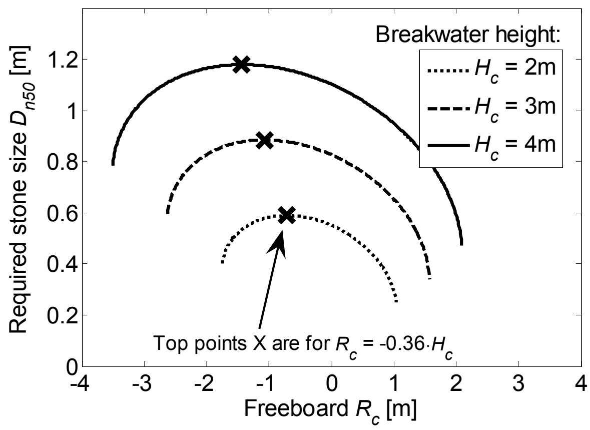


Figure 5.15. Design graphs for stability of low crested breakwaters corresponding to initiation of damage and depth limited waves with H_s/h = 0.6 and Δ = 1.6. Curves are given by Eq (5.1).

For other values of H_S/h, a similar procedure can be applied. The bathymetry in the vicinity of the structure influences the wave height distribution at the location of the structure. Generally the foreshore slope and the local water depth are the key parameters in estimating the maximum possible wave heights by the structure as the significant wave height is limited by the water depth. The depth limitation is described by Eq (5.4).

$$H_s = \gamma h = \gamma (H_c - R_c) \qquad \text{Eq (5.4)}$$

where γ is a factor depending on the foreshore slope and wave steepness

Eq (5.1) together with $\Delta = 1.6$ is used to evaluate the worst water level condition. The relative freeboard R_c/H_c is strongly dependent on the chosen γ-value. An increase in this value will allow higher waves in shallow water giving minimum stability for a larger submergence. This effect is shown in Figure 5.16 (left). The required stone size corresponding to the critical relative submergence can be found from Figure 5.16 (right).

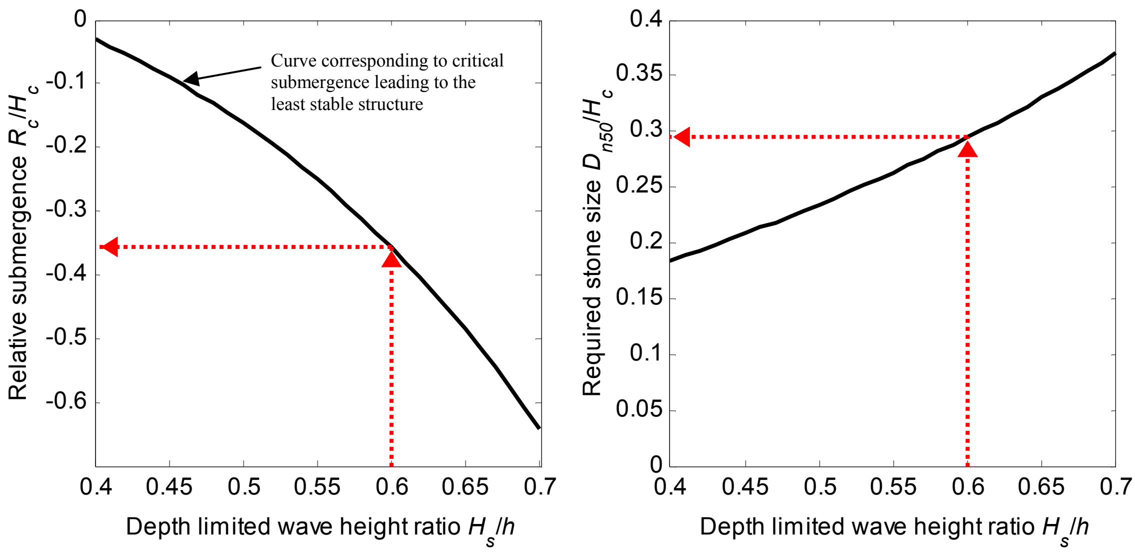


Figure 5.16. Design graphs according to Eq (5.1) for $\Delta = 1.6$. The red arrows indicates depth-limited conditions with $H_s/h = 0.6$. Left: Relative submergence corresponding to minimum stability. Right: Required stone sizes corresponding to minimum stability.

The mathematical equation used for Figure 5.16 is described in detail for general use with other values of Δ. Since the failure zone in Figure 5.17 is the convex domain above Eq (5.1), stability is assured for all water levels and for one level it is just at start of damage condition if Eq (5.4) is tangent to Eq (5.1), i.e. if the discriminant of the combined Eq (5.1) and Eq (5.4) second order equation is zero, from which:

$$\frac{D_{n50}}{H_c} = \frac{\gamma/\Delta}{1.36 - (\gamma/\Delta - 0.23)^2/(4 \cdot 0.06)} \qquad \text{Eq (5.5)}$$

Results of Eq (5.5) for different values of γ and $\Delta = 1.6$ are reported in Figure 5.16 (right) and Table 5.6. The minimum stability for a given stone size occurs at slightly submerged conditions, i.e. negative R_c. The value of γ at a given breakwater location may be estimated with a good accuracy by the method described in Chapter 6.

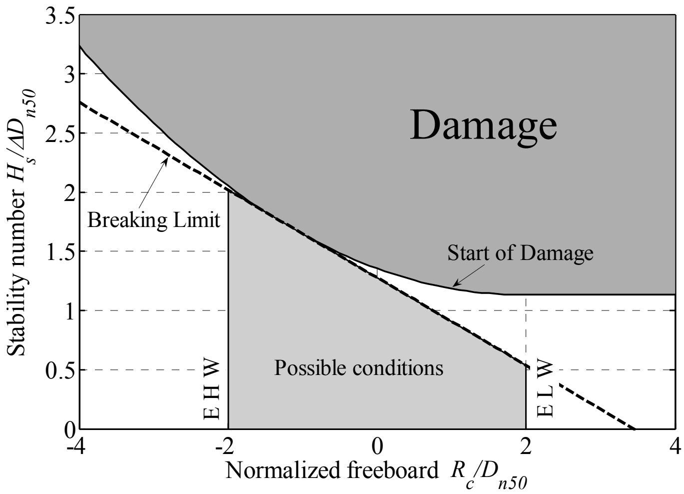


Figure 5.17. Stability condition in depth limited waves. Solid line is Eq (5.1), dashed line is Eq (5.4) scaled with ΔD_{n50} and satisfying Eq (5.5) for $\gamma = 0.6$ and $\Delta = 1.6$; its slope is γ/Δ and its intersection with the x-axis represents zero water depth conditions $R_c = H_c$. An example of Extreme High Water condition (EHW) and Extreme Low Water condition (ELW) is shown.

Table 5.6. Minimum stability for different foreshore slopes. Foreshore slope is evaluated according to the method described in Chapter 6 for s_{0p} = 0.03.

Foreshore slope	$\gamma = \frac{H_s}{h}$	$\frac{R_c}{H_c}$	$\frac{D_{n50}}{H_c}$	$\frac{H_s}{\Delta D_{n50}}$
1:∞	0.40	-0.02	0.18	0.8
1:200	0.45	-0.08	0.21	0.9
1:100	0.50	-0.16	0.23	1.5
1:40	0.55	-0.25	0.26	1.6
1:20	0.60	-0.36	0.29	1.7
--	0.65	-0.48	0.33	1.8
--	0.70	-0.64	0.37	1.9

5.7 Comparison of new and existing design curves

The AAU 2002 experiments showed basically the same overall behaviour as the NRC 1992 tests, i.e. the trunk crest was the least stable part under submerging conditions, and the leeward part of the roundhead was the least stable part in case of emergent conditions. If the same stone type is used in all sections the following rules for design can be given:

- $R_c \leq 0$, submerging conditions: The crest is the least stable part, the more submerging the more stable. Existing 2D tests and formulae for trunk armour layer stability of LCS's can be used in the design of the armour layer for the whole structure.
- $R_c > 0$, emergent conditions: Leeward part of the roundhead is the least stable, the more emergent the less stable. It is therefore on the safe side to design the roundhead according to existing knowledge about stability of roundheads for non-overtopped breakwaters.

The design curves for the least stable sections given by Vidal *et al.* 1995 (design curves for leeward head and crest given in Figure 5.2), Burger 1995 (design curve for crest damage shown in Figure 5.4), and Kramer *et al.* 2003 (design curve for least stable section given in Figure 5.10) are compared in Figure 5.18.

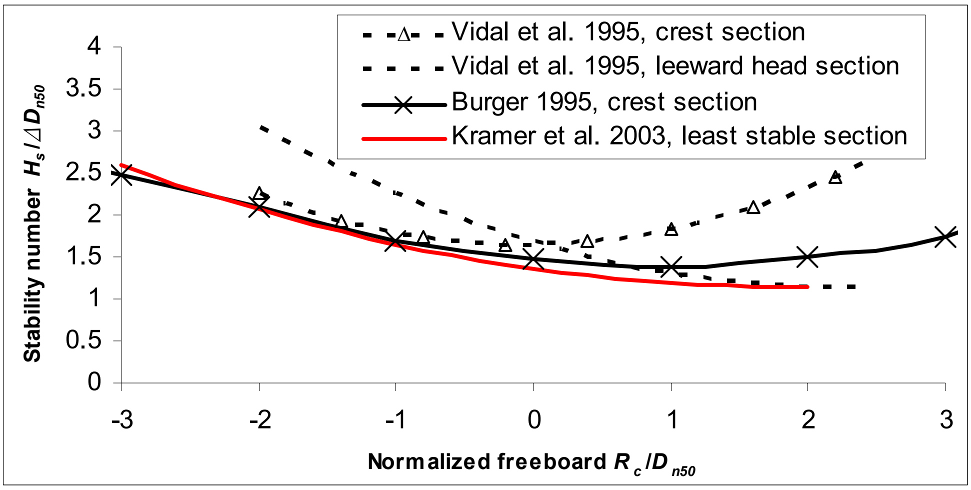


Figure 5.18. Comparison of new and existing design curves for initiation of armour damage.

The design curves shown in Figure 5.18 are in good agreement. For submerging conditions ($R_c/D_{n50} < 0$) the design curves given by the three researchers for the crest follows each other giving the same stability number for a certain freeboard. Under emergent conditions ($R_c/D_{n50} > 0$) the curves for the leeward head by Vidal *et al.* (1995) and Kramer and Burcharth (2003) gives approximately the same stability number. Design by the single formula provided by Kramer and Burcharth (2003) will therefore be safe.

5.8 Validation of the new stability formulae with prototype experience

The rule of thumb and Eq (5.1) have been validated with information about the breakwaters described in Table 5.7 and a good agreement was found. All breakwaters in the DELOS inventory for which the required parameters were available have been included in the list. In three cases armour damage was experienced (*IT Ostia 1990*, *IT Sirolo*, *IT Scossicci*). This is in agreement with the formulae as these three cases do not satisfy Eq (5.1).

For the low structures in Table 5.7 (H_c < 4 m) the same rock type, crest width and slopes are used in trunk and roundhead sections. Design condition is depth limited waves under submerged conditions, which in most cases corresponds to the highest design water level. For the submerged ($R_c \leq$ -1 m) and very low ($H_c \leq$ 3 m) structures the design water depth is during normal water level conditions or even for the lowest design water level. This is for example the case for *ES Paolo*, for which $h_{design,\ lowest}$ = 3.8 m.

For the high structures ($H_c \geq$ 4 m) wider crests and/or less steep slopes are used in the roundhead. This is the case for *UK Elmer*, *GR Lakopetra*, and *GR Paphos*. At *ES Altafulla* a wider roundhead with larger rocks are used.

Table 5.7. Existing EU breakwater designs. RoT is "Rule of Thumb" given by Eq (5.2).

	Armour size D_{n50} [m]	Structure height H_c [m]	Freeboard R_c (MWL) [m]	Water depth h (MWL) [m]	$\frac{H_c}{D_{n50}}$	Satisfies RoT	Satisfies Eq (5.1)
DK, Lønstrup	0.80	2.3	+1.3	1.0	2.9	√	√
DK, Skagen	0.71	2.0	+1.0	1.0	2.9	√	√
GR, Lakopetra	1.00	4.0	+0.7	3.3	4.0	÷[1)]	√
GR, Alaminos	1.10	3.5	+0.5	3.0	3.1	√	√
GR, Paphos	1.40	4.5	-0.3	4.8	3.2	√	√
UK, Elmer	1.45	6.0	+4.3	1.7	4.1	÷[2)]	÷[2)]
UK, Monk's Bay	1.31	3.7	+2.2	1.5	2.8	√	√
ES, Altafulla	1.31	4.5	+0.5	4.0	3.4	√	√
ES, Comin	0.87	3.0	+0.5	2.5	3.4	√	√
ES, Postiguet	0.57	2.0	-2.0	4.0	3.5	√	√
ES, Palo	0.91	2.8	-1.5 to -2.0	4.3 to 4.8	3.1	√	√
IT, Punta Marina	0.90	2.8	-0.2	3.0	3.1	√	√
IT, Lido di Dante	0.80	2.5	-0.5	3.0	3.1	√	√
IT, Cesenatico	0.90	2 to 2.5	-0.5	2.5 to 3.0	2.2 to 2.8	√	√
IT, Ostia (1990)	0.65	2.5	-1.5	4.0	3.9	÷	÷[3)]
IT, Ostia (2003)	0.90	3.0	-1.0	4.0	3.3	√	√
IT, Sirolo	0.90	2.5 to 4.0	-1.0	3.5 to 5.0	2.8 to 4.4	÷	÷[4)]
IT, Scossicci	0.99	4.20	-1.0	5.20	4.2	÷	÷[4)]
IT, Grottammare	0.90	1.6	-0.9	2.5	1.8	√	√
IT, Bisceglie	1.04	2.55 to 4.15	-0.15	2.7 to 4.3	2.5 to 4.0	(÷)[5)]	√
IT, Nettuno	0.86	2.5	-0.5	3.5	2.9	√	√
IT, Amendolara	1.36	2.3	-0.5	2.8	1.7	√	√
IT, Pellestrina	0.76	2.5	-1.5	4.0	3.3	√	√

Notes:

1) GR, Lakopetra: $H_{s,\ design}$ = 2.4 m occurring during the design water depth $h \cong 4$ m corresponding to approximately zero freeboard. For this event N_s = 1.4, which satisfies Eq (5.1).

2) UK, Elmer: Extreme high water depth h = 5.4 m corresponding to freeboard R_c = +0.6 m. The maximum significant wave height is estimated as $H_s = 0.6 \cdot h$ = 3.2 m corresponding to N_s = 1.4. This is slightly more than the stability number calculated by Eq (5.1). The Elmer structures have gentle slopes of 1:2.5 and wider roundheads, which makes the structures more stable than calculated by Eq (5.1).

3) IT, Ostia: Over a decade (1990 – 2003) reshaping was experienced resulting in crest lowering of about 0.5 m. Damage to the structures was in the range 4 % to 25 %. In 2003 the structures were therefore recharged and raised to R_c = -1.0 m with larger rocks. The 1990 breakwaters did not satisfy the rule of thumb.

4) IT, Sirolo and Scossicci: Damage to some structures experienced. Some structures have been rebuilt. The breakwaters does not satisfy the rule of thumb and Eq (5.1).

5) IT, Bisceglie. $H_{s,\ design}$ = 2.8 m occurring during the design water depth h = 5.1 m corresponding to freeboard R_c = -1.0 m. For this event N_s = 1.6, which satisfies Eq (5.1).

When no notes about damage are given the structures have not showed any sign of damage.

5.9 Residual stability and damage development

The following formulae were based on laboratory tests with 2D-irregular, head-on waves. Real LCS's will usually be designed for depth-limited 3D-waves, which are more damaging to the structure. The following formulae are therefore expected to underestimate the required rock-size, and caution should therefore be taken if the formulae are used for design in such conditions. However, the formulae are very useful to evaluate the residual stability if some reshaping and crest-lowering of the breakwater is allowed.

Van der Meer (1990) formula, reef breakwaters

The formula was established for the trunk of low-crested reef homogeneous breakwaters described by Figure 5.19. The formula was based on laboratory tests with 2D-irregular, head-on waves. Data by Ahrens (1987) and Van der Meer (1990) was used.

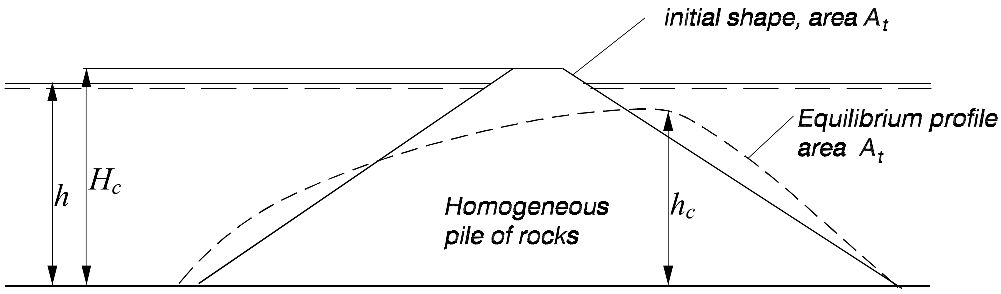


Figure 5.19. Definition sketch for reshaping reef breakwaters.

The equilibrium height of the structure (irregular, head-on waves) is given by:

$$h_c = \sqrt{\frac{A_t}{\exp(aN_s^*)}} \qquad \text{with a maximum of } H_c \qquad \text{Eq (5.6)}$$

where A_t area of initial cross section of structure
h water depth at toe of structure
H_c initial height of structure
N_s^* spectral stability number, $N_s^* = \frac{H_s}{\Delta D_{n50}} s_{0p}^{-1/3}$
$a = -0.028 + 0.045 \frac{A_t}{H_c^2} + 0.034 \frac{H_c}{h} - 6 \cdot 10^{-9} \frac{A_t^2}{D_{n50}^4}$

No ranges of the parameters in Eq (5.6) were given by Ahrens or Van der Meer. However, Eq (5.6) seems only to be valid for fairly narrow structures. This is explained further. For structures with wider crests (i.e. larger area A_t) the required stone size is larger, given that the crest lowering is fixed. This is not in agreement with the physics (a wider structure should be at least as stable as a narrow one). Van der Meer tested a structure with $0.5 \le B/H_c \le 1$ (B is crest width). It is therefore assumed that the equation is only valid for fairly narrow structures as indicated by the shape of the sketch in Figure 5.19.

Van der Meer (1991) formula, submerged breakwaters

The formula given in Eq (5.7) was established for the trunk of submerged breakwaters with two-layer armour. The formula was based on laboratory tests with regular and some 2D-irregular, non depth-limited, head-on waves. Data were used from Givler and Sorensen (1986) who performed tests with regular head-on waves on a trunk-section with slope 1:1.5; and Van der Meer (1991) who performed tests with irregular head-on waves on a trunk-section with slope 1:2.

$$\frac{H_c}{h} = (2.1 + 0.1S)\exp(-0.14N_s^*) \qquad \text{Eq (5.7)}$$

where h water depth
H_c height of structure over sea bed level
S relative eroded area
N_s^* spectral stability number, $N_s^* = \frac{H_s}{\Delta D_{n50}} s_{0p}^{-1/3}$

Typical example of damage development in trunks and roundheads, Kramer *et al.* (2003)

Kramer *et al.* (2003) showed that the leeward part of the roundhead is the most exposed part of the breakwater for emerged conditions. For submerged conditions the trunk crest is the most exposed part. An example of the test data for emerged conditions (R_c/D_{n50} = +1.5) and submerged conditions (R_c/D_{n50} = -1.5) is shown in Figure 5.20. The shown test results are for head-on 3D waves with wave steepness s_p = 0.02.

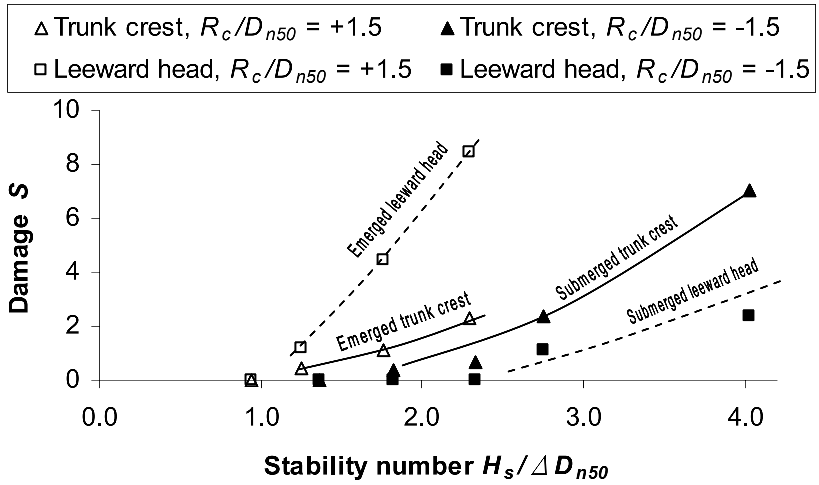


Figure 5.20. Typical example of damage development for test results from the 3-D AAU tests. The lines indicate the trend of the data; dashed lines are for leeward head and full lines are for trunk crest.

From Figure 5.20 it is seen that the structure is most vulnerable under emerged conditions as the unfilled markers in the figure corresponds to larger damage than the filled markers. Further it is observed that the leeward head is the most exposed part for emerged conditions but the most stable part for submerged conditions. For emerged conditions the progress of the damage of the leeward head is much more rapid than for the trunk crest (the slope of the left line in the figure is much steeper than the others), meaning the difference in stability numbers between initiation of damage and complete destruction is small. For emerged conditions the selection of proper safety margins for the roundhead is therefore important as exceedance may lead to quick destruction. If design condition is for submerged conditions then less strict safety factors are necessary.

The result is well in agreement with the way existing LCS's are designed. From Table 5.7 it was concluded that low regularly overtopped breakwaters have the same rock type, crest width and slopes in trunk and roundhead sections. For the high emerged breakwaters wider crests, larger rocks and/or less steep slopes are used in the roundhead.

5.10 Example of required stone size according to the formulae and diagrams

In Table 5.7 it is seen that the height of a typical LCS cross-section is about H_c = 2 to 4 m. In this example a cross-section height H_c = 3 m, slopes 1:2 and a crest-width of 3 m is used. Rock with submerged density Δ = 1.6 is applied. Two conditions with depth-limited wave attack are investigated:

1) Water depth h = 3 m corresponding to zero freeboard (R_c/H_c = 0)
2) Water depth h = 4 m corresponding to freeboard R_c = -1.0 m (submerged conditions with R_c/H_c = -0.33).

The question is: What is the required stone size according to the formulae to resist the conditions?

The significant wave height is estimated as $H_s = 0.6 \cdot h$, and a wave steepness s_{0p} = 0.02 is used in the Van der Meer 1990 and 1991 formulae.

Table 5.8. Example of required stone size according to armour stability formulae for a typical structure with height H_c = 3 m. Depth-limited waves. ID is Initiation of Damage.

Formula	Damage	Required stone size	
		Zero freeboard condition (h = 3.0 m)	Submerged condition (h = 4.0 m)
Rule of thumb	ID	0.90	0.90
Eq (5.1)	ID	0.83	0.88
Burger (1995)	$S = 2$	0.70	0.83
Van der Meer (1991) formula	$S = 0$	0.78	0.75
	$S = 2$	0.70	0.69
	$S = 5$	0.61	0.62
Van der Meer (1990) formula	$h_c = H_c$	0.53	0.67
	$h_c = 0.9H_c$	0.45	0.56
	$h_c = 0.8H_c$	0.38	0.47
Vidal (1992), trunk	$S = 1.5$	0.70	0.70
Based on curves given	$S = 2.5$	0.60	0.60
in Appendix F.4	$S = 6.5$	0.45	0.45

From the example given in Table 5.8 the following can be concluded:

- According to the van der Meer 1990 formulae a smaller stone size can be used if a homogeneous cross-section is used.
- If some reshaping resulting in crest lowering is allowed the required nominal stone diameter can be reduced by 20-40%.
- The required stone size by the different methodologies varies significantly. The trend seems to be that formulae developed mainly by use of regular non depth-limited 2D waves gives the smallest required stone size, whereas the formulae developed with 3D irregular depth-limited breaking waves leads to the largest required stone size.

Please note: The tests with non depth-limited 2D waves is expected to lead to an underestimation of the required rock size for the conditions in Table 5.8. It is therefore recommended to use the results from Table 5.8 only for comparisons to evaluate residual stability and not for design of LCS's in depth-limited 3D waves.

6 Wave heights in shallow water

At the time of preliminary design of the LCS the designer usually only have limited knowledge about the wave climate. The local wave climate at the LCS location is generally unknown, but the wave characteristics in deep water offshore are often known.

Wave transformation from deep to shallow water is generally complicated due to wave shoaling, refraction, diffraction, breaking and energy dissipation. Numerical models can be used to calculate a local wave climate, but such models normally require detailed bathymetric maps and are costly in time and money. Therefore more simple models are often used to calculate local wave characteristics. Such simple models are normally based on wave theories taking account of the wave transformation phenomena described above in combination with empirical models based on model tests.

The most commonly used simple methods today are the ones by Goda (1985) and Battjes and Groenendijk (2000). The wave transformation model by Battjes and Groenendijk (2000) is an effective and simple model for calculation of wave height distributions on shallow foreshores, and it takes account for the local water depth and foreshore slope. The model has been validated by the measured waves in the 3D-stability tests at AAU and a good agreement was found, see Appendix D. As the proposed stability formulae Eq. 4.1 and Eq.4.2 are developed using the measured waves at the foot of the structures, the model by Battjes and Groenendijk (2000) is a good and effective tool to combine with the the proposed stability formulae. The mathematics is described in Battjes and Groenendijk (2000). It is chosen here to include a simple model, which is based on lookups in diagrams and tables. The diagrams were produced by Van der Meer, and they can be found in Crossman *et al.* (2003).

Significant wave heights H_{m0} and H_{s0} are almost equal in deep water. However, in shallow water they differ. Based on known local water depth h, beach slope, and deepwater waves with H_{m0} and T_p the local significant wave H_s at the breakwater location can be estimated.

Inputs:

H_{m0} and T_p in deep water

h: Local water depth

m: Foreshore slope

→

Output:

H_s at the local water depth

The following procedure can be applied:

1) Calculate the deep water wave length by $L_{0p} = gT_p^2 / 2\pi$
2) Calculate inputs for Figure 6.1: $s_{0p} = H_{s0} / L_{0p}$ and h / L_{0p}
3) Use the two inputs and the foreshore slope m to find the local ratio H_{m0} / h in Figure 6.1
4) Use the local H_{m0} / h and water depth h to calculate $H_{rms} = 0.67 H_{m0} + 0.215 H_{m0}^2 / h$
5) Calculate $H_{tr} = (0.35 + 5.8 \tan m) \cdot h$, where *"tan m"* is the foreshore slope
6) Use 4) and 5) to calculate $\overset{\approx}{H}_{tr} = H_{tr}/H_{rms}$ (≈ denotes normalized value)
7) Use $\overset{\approx}{H}_{tr}$ in Table 6.1 to find $\overset{\approx}{H}_{1/3}$
8) By using 4) and 7) calculate the local significant wave height $H_s = H_{rms} \cdot \overset{\approx}{H}_{1/3}$

Table 6.1. Characteristic dimensionless wave heights. ≈ denotes normalized value with respect to H_{rms}, Crossman *et al.* (2003).

$\tilde{\tilde{H}}_{tr}$	0.05	0.5	1	1.2	1.35	1.5	1.75	2	2.5	3
$\tilde{\tilde{H}}_{1/3}$	1.279	1.28	1.324	1.371	1.395	1.406	1.413	1.415	1.416	1.416

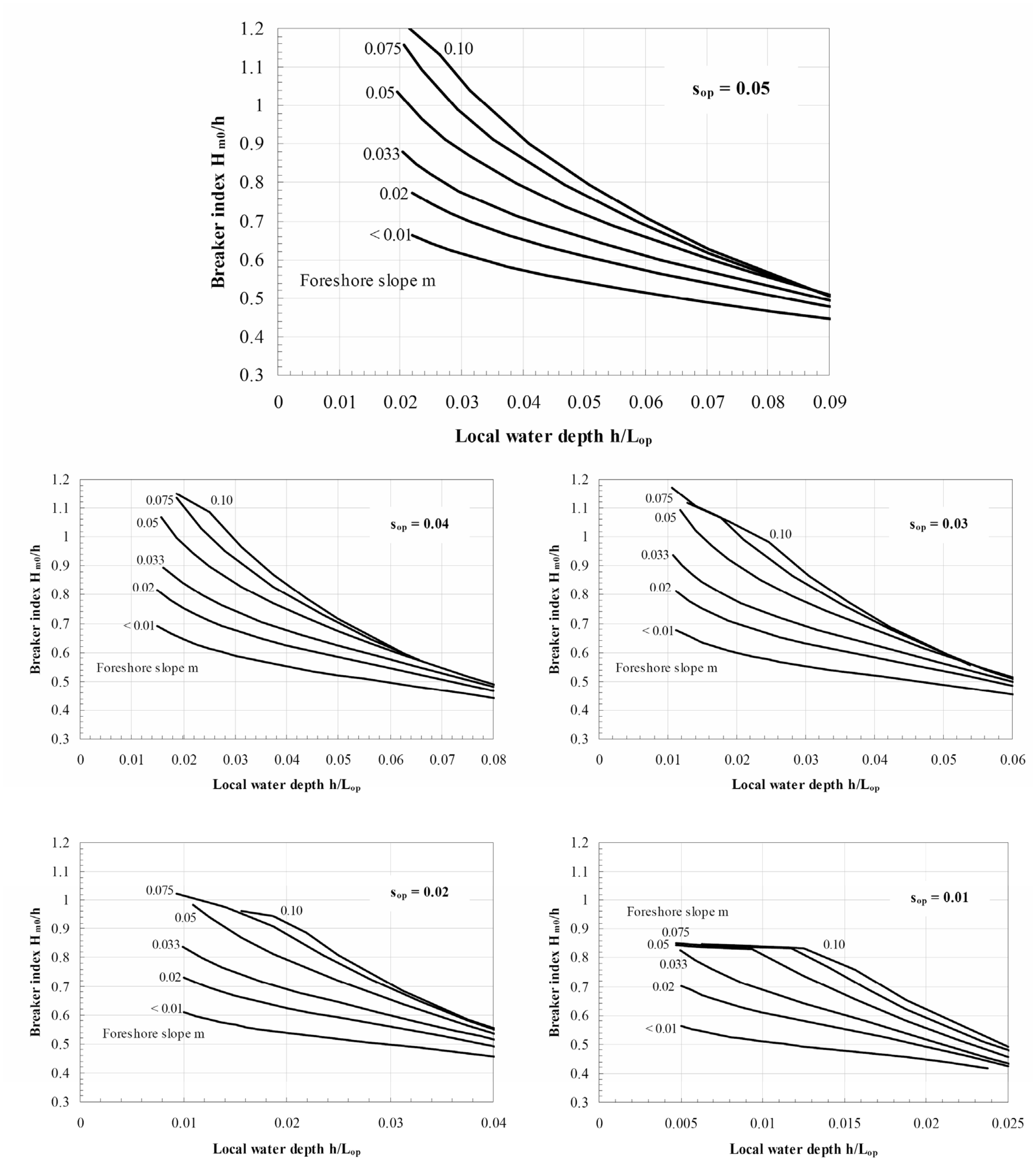


Figure 6.1. Shallow-water significant wave heights for uniform foreshore slopes, Crossman *et al.* (2003)

In case of oblique waves similar diagrams can be found in Burcharth and Lamberti (2006).

7 Bedding layer and geotextiles

Subsidence of the armour into the sea bed is prevented by a bedding layer and/or geotextiles. A bedding layer helps to distribute the structure's weight over the underlying base material to provide more uniform settlement. Granulated filters are commonly used as a bedding layer on which a coastal structure rests. It is advisable to place coastal structures on a bedding layer (along with adequate toe protection) to prevent or reduce undermining and settlement. When rubble structures are founded on cohesionless soil, especially sand, a bedding layer should be provided to prevent differential wave pressures, currents, and groundwater flow from creating an unstable foundation condition through removal of particles. Even when a bedding layer is not needed in the completed structure, bedding layers may be used to prevent erosion during construction to distribute structure weight or to retain and protect a geotextile filter cloth.

Placing larger armour stone directly on geotextile filter cloth is likely to puncture the fabric either during placement or later during armour settlement. Placing a bedding layer over the geotextile fabric protects it from damage. In this application there is more flexibility in specifying the bedding layer stone gradation because the geotextile is retaining the underlying soil.

The design of geotextiles and bedding layers in relation to LCS's follows the same procedures as for conventional breakwaters. For in depth guidance on the use and design of geotextiles the reader is referred to standard literature, e.g. Pilarczyk (2000) and PIANC (1992).

8 Toe berm stability

The function of a toe berm is to support the main armour layer and to prevent damage resulting from scour. Armour units displaced from the armour layer may come to rest on the toe berm, thus increasing toe berm stability. Toe berms for LCS's are normally constructed of quarry rock, concrete blocks are to the authors knowledge so far not used for LCS's, but can be used if quarry rock material is too small or unavailable.

Scour at the base of the structures causes high level of disturbance to the ecological communities, leading to increased mortality, especially for filter feeders such as barnacles and algae. This effect can be minimised by building a highly stable berm around the structures, particularly on the seaward side or by providing more refugia such as crevices and holes.

In shallow water with depth-limited design wave heights, support of the armour layer at the toe is ensured either by placing one or two extra rows of main armour units at the toe of the slope or by the use of stones or blocks in the toe that are smaller than the main armour, c.f. examples given in Figure 1.4 and Figure 1.5. These solutions are stable provided that scour does not undermine the toe causing the armour layer to slide. The toe berm must be wide enough to avoid this problem, which is treated in the chapter subsequent dealing with scour.

Toe berm stability is mainly affected by wave height, water depth at the top of the toe berm, width of the toe berm, and block density. However, wave steepness does not appear to be a critical toe berm stability parameter.

Model tests with irregular waves indicate that the most unstable location is at the shoulder between the slope and the horizontal section of the berm. The instability of a toe berm will trigger or accelerate the instability of the main armour. Lamberti (1995) showed that moderate toe berm damage has almost no influence on armour layer stability, whereas high damage of the toe berm severly reduces the armour layer stability. Therefore, in practice it is economical to design toe berms that allow for little damage.

No model tests dealing especially with toe berm stability of LCS's exist. However, within DELOS a few model tests on LCS's with depth limited waves and wave breaking at the toe showed good agreement with the formula for trunk toe stability of emerging breakwaters given by Eq (8.1). Description of the test setup can be found in Kramer *et al.* (2005). For LCS's wave energy can pass over the structure making them more stable than the conventional type. Seaward toe berms designed by formulae developed for non overtopped breakwaters will therefore be more stable when used for LCS's. This was confirmed by the model tests performed within DELOS. The tests showed that the seaward toe was more prone to damage than the leeward toe. This indicates that it is safe to apply the same stone type in the leeward toe as used for the seaward toe. Further the DELOS testing showed that oblique wave attack was less damaging than normal incidence wave attack.

8.1 Toe berm stone sizes in trunk

The formula by Van der Meer *et al.* (1995) given in Eq (8.1) may be used to find the required rock size for the toe berm for the trunk. The formula was developed for sloping, emergent rubble mound breakwaters. Stones having a mass density of 2.68 t/m^3 were used, and the berm width was varied.

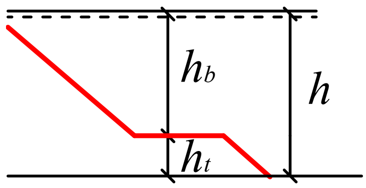
$$N_s = \frac{H_s}{\Delta D_{n50}} = \left(0.24\frac{h_b}{D_{n50}} + 1.6\right) N_{od}^{0.15} \qquad \text{Eq (8.1)}$$

where

H_s	Significant wave height in front of breakwater
Δ	Relative mass density $\Delta = (\rho_s/\rho_w)-1$
ρ_s	Mass density of stones
ρ_w	Mass density of water
D_{n50}	Equivalent cube length of median stone
h_b	Water depth at top of toe berm
N_{od}	Number of units displaced out of the armour layer within a strip width of D_{n50}. For a standard toe size of about 3-5 stones wide and 2-3 stones high:

$$N_{od} = \begin{cases} 0.5 & \text{no damage} \\ 2 & \text{acceptable damage} \\ 4 & \text{severe damage} \end{cases}$$

For a wider toe berm, higher N_{od} values can be applied.



The formula is valid for:
Irregular head on waves; nonbreaking, breaking and broken waves
$0.4 < h_b / h < 0.9$, $\quad 0.28 < H_s / h < 0.8$, $\quad 3 < h_b / D_{n50} < 25$

If the highest waves are depth limited then the significant wave height may be replaced by the approximation $H_s = 0.6 \cdot h$. By inserting in Eq (8.1) $\rho_s = 2.65$ t/m^3 corresponding to $\Delta = 1.6$, and $H_s = 0.6 \cdot h$, Eq (8.1) can be reduced to:

$$N_{od} = 0.5: \quad \begin{cases} D_{n50} = 0.16 \cdot h & , h_t = 2 \cdot D_{n50} \\ D_{n50} = 0.20 \cdot h & , h_t = 3 \cdot D_{n50} \end{cases}$$
$$N_{od} = 2: \quad \begin{cases} D_{n50} = 0.09 \cdot h & , h_t = 2 \cdot D_{n50} \\ D_{n50} = 0.11 \cdot h & , h_t = 3 \cdot D_{n50} \end{cases} \qquad \text{Eq (8.2)}$$

However, if the toe is located in very shallow water and the toe is expected to be very exposed to direct wave action, then the same stone type as used in the armour layer can be applied. This will always lead to a stable conservative design.

During the 2D tests at AAU 2005 described in chapter 5.3 the breakwater was exposed to maximum depth limited waves in water depths h = 4 to 34 cm. No displacements of the toe stones took place. The parameter intervals for the tests are within the valid intervals for Eq (8.1) except that also values of h_b / D_{n50} down to zero were tested. Using the test conditions, Eq (8.1) gives N_{od} = 0 to 0.10, which corresponds to no displacements, see Table 8.1. Thus Eq (8.1) predicts correctly the tests results.

Table 8.1. Conditions for toe berm in the 2D tests at AAU 2005.

h_b/D_{n50}	h_b/h	H_s/h	N_s in tests	N_{od} according to Eq (8.1)
0.00	0.00	0.51	0.35	0.000
1.39	0.56	0.50	0.76	0.002
2.50	0.69	0.47	1.04	0.007
3.62	0.76	0.46	1.32	0.016
4.45	0.80	0.46	1.56	0.028
5.29	0.83	0.45	1.75	0.038
6.12	0.85	0.43	1.90	0.041
7.23	0.87	0.43	2.17	0.057
8.35	0.88	0.45	2.56	0.103

8.2 Toe berm stone sizes in roundheads

For the toe berm in the roundhead no specific recommendations exist. In many situations previous experiences can be used to evaluate the necessary size of the rocks. Rock sizes equal to the sizes by the trunk might be used, but in that case it is recommended to validate the design by the use of model tests. If the LCS structures are long and low very large rip currents might occur in the gaps. This might affect the toe stability especially if scour takes place in front of the toe. If model tests are used to design the toe berm it is very important that the rip currents are modelled correct in the experiments.

If the toe is located in very shallow water and the toe is expected to be very exposed, then the same stone type as used in the main armour layer of the roundhead can be applied. This will always lead to a stable conservative design.

9 Scour protection

As the author has not performed any tests or further research on scour protection for LCS's, reference about scour protection design is given to the work by Sumer and Fredsøe, e.g. as described by Sumer *et al.* (2005) and Burcharth and Lamberti (2006). A brief introduction based on these references is included below.

It is imperative to construct a protection layer for toe protection. This protection layer may be constructed in the form of a protection apron. The apron must be designed so that it will remain intact under wave and current forces, and it should be "flexible" enough to conform to an initially uneven seabed. With this countermeasure, scour can be minimized, but not entirely avoided. Some scour will occur at the edge of the protection layer, and consequently, toe stones will slump down into the scour hole. This latter process will, however, lead to the formation of a protective slope, a desirable effect for "fixing" the scour. The determination of the width of the protection layer is an important design concern. The width should be sufficiently large to ensure that some portion of the protection apron remain intact, providing adequate protection for the stability of the breakwater, see Figure 9.1.

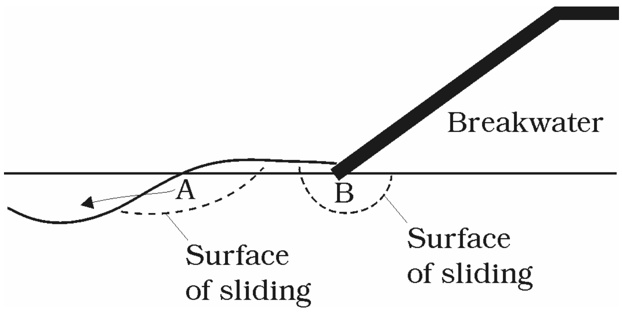


Figure 9.1. Possibility of sand slide in front of breakwater.

It is worth to underline that the above design formulae for armour and toe design are empirically derived from fixed-bed models. In real field conditions the seabed foundation is typically highly mobile, consisting in most cases of fine sands subjected to intense hydrodynamic action.

Indeed observed prototype damage of LCS's, as described by Lamberti *et al.* (2005) and Burcharth *et al.* (2006), is often the consequence of geotechnical or morphodynamic instabilities rather than hydraulic response. First of all, the simple dumping of stones onto the sand bed causes sinking and settlements, especially when proper bottom protection is not used. Moreover, breaking waves and the related strong nearshore currents tend to produce local scour and deposition near the structure toe, which affect the toe stability either directly (e.g. scour sliding) or indirectly (variation in local water depth and thus changes of incident wave height). Finally, sand intrusion and infilling plus ecological colonization can strongly reduce the structure porosity and wave energy absorption properties.

Generally to avoid sinking of the rubble stone material into a sandy seabed it is necessary to separate the two materials by the use of small stone filter layers, geotextiles or mattresses.

10 Wave transmission

The function of LCS's is to reduce wave energy in their lee. The structures reduce the incoming wave energy across the structure by triggering wave breaking at and on the structure, by partially reflecting the waves, and by dissipation related to the wave induced porous flow in the structure. This is illustrated for an emergent structure in Figure 10.1. The ratios between the three types of energy components depend on structure geometry, freeboard and wave characteristics.

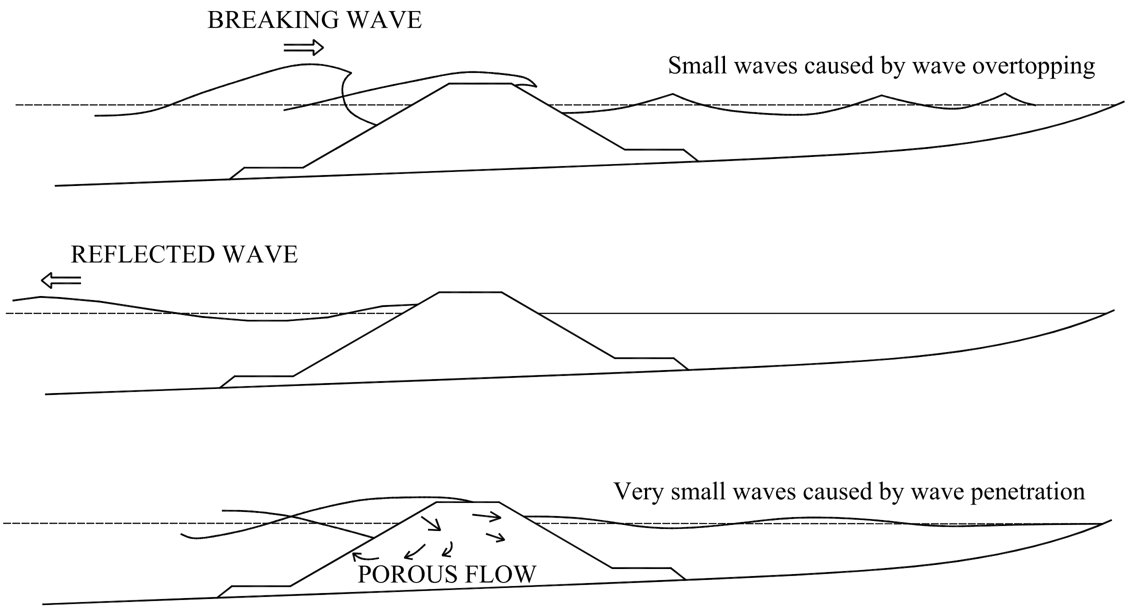


Figure 10.1. Illustration of the sheltering effect of an emergent LCS by reduction in shorewards transmitted wave energy by wave breaking, wave reflection and porous flow.

Most commonly wave breaking accounts for the largest part of the energy reduction, reflection for the second largest part, and porous flow for the smallest part. Wave energy is also transmitted horizontally by diffraction and refraction around the heads of the structure into the lee zone. The wave transmission generates a water level set-up behind the structures, which in case of submerged structures generates large rip currents in the gaps as illustrated in Figure 10.2.

Low-crested breakwaters are generally fairly short (the length is typically 25 to 100 m, see Figure 2.3 in Chapter 2), making horizontal wave transmission important. In the case of shorter emergent structures with only limited overtopping the horizontal wave transmission will dominate. Thus the wave agitation in the lee of the structure is mainly caused by diffraction and refraction of waves at the heads of the structure. The lower the crest level the more dominant will be the wave disturbance caused by overtopping waves. For long submerged structures the wave disturbance is caused almost completely by wave transmission over the crest.

Generally wave transmission formulae only consider wave transmission over the crest, i.e. the formulae does not account for horizontal wave transmission. Some recommendations for wave transmission formulas in case of rubble mound structures are given in the following. Wave transmission characteristics are different for smooth and impermeable structures. However, as such structures are rather atypical and not a topic for this thesis the reader is referred to Kramer *et al.* (2003) and Van der Meer *et al.* (2003), and further guidance is not given here.

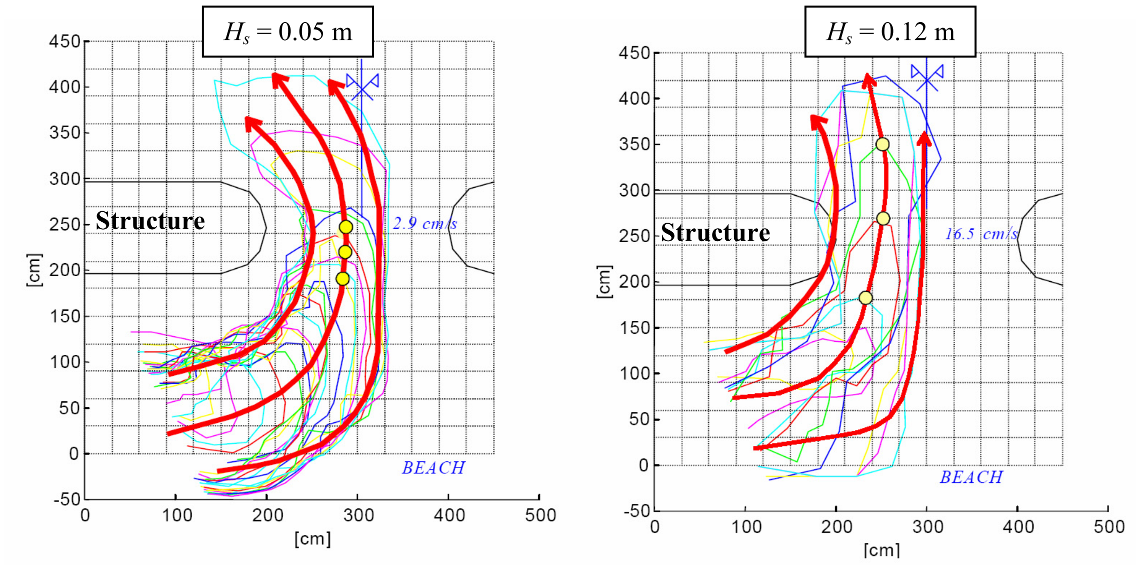


Figure 10.2. Flow patterns in the gap area, measured in a model. The red arrows indicates the flow direction. Example of results from the 3D hydrodynamic tests at Aalborg University, see Kramer *et al.* (2003).

The author has performed numerous hydrodynamic 3D tests at Aalborg University in collaboration with DELOS partners; see Kramer *et al.* (2005). The studied features were: Wave induced currents (long-shore and in the roundhead section), wave reflection and transmission, wave overtopping, pumping effects and set-up behind the structures. Some elaborated test results can be found in Kramer *et al.* (2003), Van der Meer *et al.* (2003) and Lamberti *et al.* (2003).

The 3D wave transmission test performed at Aalborg University for DELOS gave results with regard to oblique wave attack and transmission, see Appendix H. The main conclusion on the effect of wave direction was that the wave transmission coefficient was only marginally affected by wave direction within the tested ranges (up to a wave obliquity of about 60°, with 0° as perpendicular wave attack). This conclusion underlines that existing wave transmission formulas, developed for perpendicular wave attack, can also be used for oblique wave attack, at least up to 60°, i.e. all structures covered by this thesis.

The influence of short-crested waves versus long-crested waves on the wave transmission was also investigated in case of both oblique and perpendicular wave attack. Keeping in mind the conclusion about the obliquity, the conclusion was not surprising, namely that the wave transmission coefficient did not depend on whether the waves were short-crested or not.

Another question with regard to oblique wave attack is whether the transmitted wave angle is similar to the incident wave angle. The results of the tests at Aalborg University showed that the transmitted wave angle is consistently slightly smaller than the incident one (the transmitted angle was roughly 90 % of the incident angle in the tests). This result is believed to be due to refraction effects.

Diagrams and parameterized wave transmission formulae with perpendicular long-crested wave attack have been developed by numerous researchers, see Appendix H. Important parameters influencing the wave transmission are: Characteristic wave height and steepness, freeboard, crest width, but also the porosity of the structure is important. The newest work covering a wide range of structures can be found in Briganti *et al.* (2003). The main conclusion of Briganti's work is that if submerged rubble mound structures with very wide crests are considered, two formulae should be considered, one for relatively narrow crested structures and one for very wide and submerged structures. The two new formulas have similar structure as the formulas originally proposed by d´Angremond *et al.* (1996). The formulae are given by:

$$K_t = -0.4\frac{R_c}{H_s} + 0.64\left(\frac{B}{H_s}\right)^{-0.31}\left(1-e^{-0.5\xi}\right) \quad , B/H_s < 10 \qquad \text{Eq (10.1)}$$

Limits: $K_{t,\,min} = 0.075$ and $K_{t,\,max} = 0.80$

$$K_t = -0.35\frac{R_c}{H_s} + 0.51\left(\frac{B}{H_s}\right)^{-0.65}\left(1-e^{-0.41\xi}\right) \quad , B/H_s > 10 \qquad \text{Eq (10.2)}$$

Limits: $K_{t,\,min} = 0.05$ and $K_{t,\,max} = -0.006\,B/H_i + 0.93$

K_t	transmission coefficient for wave height $K_t = H_t/H_s$
H_s	incident significant wave height at the toe of the structure
H_t	transmitted significant wave height
ξ_{op}	breaker parameter $\xi_{op} = \tan\alpha/\sqrt{s_{op}}$ (α is structure slope)
s_{op}	wave steepness, $s_{op} = 2\pi H_s/(gT_p^2)$
T_p	peak wave period
R_c	crest freeboard
B	crest width

According to Eq (10.1) and Eq (10.2) the wave transmission coefficient decreases for increased crest width, i.e. a wider structure transmits less wave energy. This effect is shown in the example with zero freeboard conditions given in Figure 10.3. It is seen, that the curves for Eq (10.1) and Eq (10.2) does not give the same transmission coefficient at $B/H_s = 10$, the transmission coefficient evaluated by Eq (10.2) is much lower. To compensate for the (large) discontinuity at $B/H_s = 10$ Van der Meer *et al.* (2005) suggests using Eq (10.1) for $B/H_s < 8$, Eq (10.2) for $B/H_s > 12$, and to linearly interpolate in the range $8 < B/H_s < 12$.

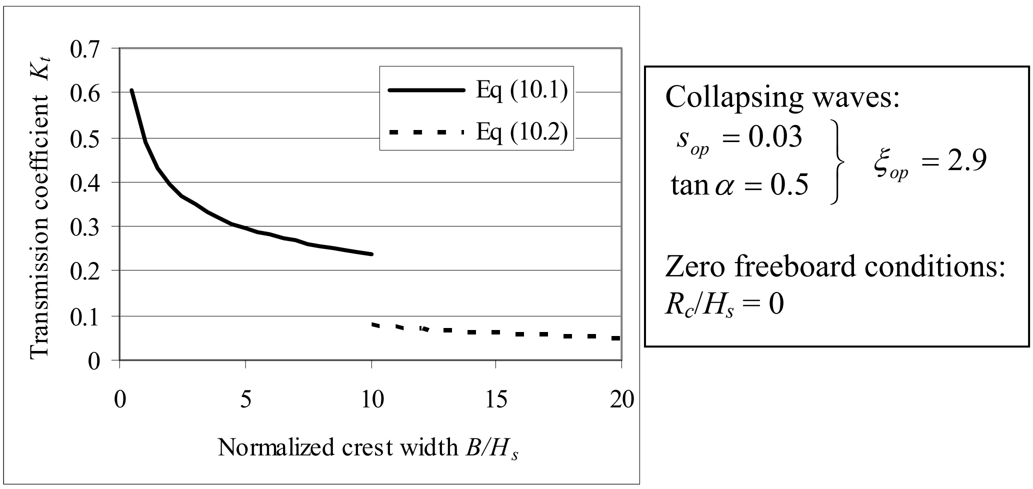


Figure 10.3. Effect of crest width according to Eq (10.1) and Eq (10.2), zero freeboard and $\xi_{op} = 2.9$.

11 Conclusions and recommendations

A new design formula corresponding to the state of damage initiation for rock armoured low-crested structures exposed to shallow water waves is presented. The formula is valid for armour slope 1:2 and is based on 3-D model tests in short-crested waves. In case of depth limited waves slightly submerged conditions are the most critical with respect to armour stability. Corresponding to such conditions a simple rule of thumb is presented according to which the nominal diameter for rock with mass density 2.65 t/m^3 should be approximately 20 to 30 % of the height of the structure, dependent on foreshore slope and wave steepness. This rule has been validated against new 2-D model tests and performance of several prototype structures examined in DELOS through the LCS inventory.

The validity of an existing toe-berm stability formula has been verified by comparison to a few new model tests in depth limited waves, and a good agreement was found also at very low water depths.

Stones used in the armour layer of a LCS must be sufficiently large to avoid undesirable displacements caused by the wave action against the structure. As LCS's are built in shallow water the highest waves will often be depth limited. As a consequence the structures will typically be exposed to design waves numerous times during the lifetime. Because damage is cumulative it is important to design such structures for low damage criteria. Moreover, because narrow-crested breakwaters built in shallow water are only a few stone-sizes high and wide, one stone removed from the edge of the crest will cause a relatively large hole in the cross-section leading to increased wave transmission. Stability must be ensured to some degree, and the author recommends using fairly large stones compared to current tradition. The recommendations are not only based on existing knowledge about failed structures, but using fairly large stones helps minimizing the following problems, which are typical for LCS's.

1) Lack of knowledge or resources to adequately maintain structures (e.g. regular maintenance and inspection is expensive and requires competent and experienced expertise)
2) Reduced environmental impact / damage (static stable structures are preferred from an ecological point of view, due to their lower impact on the habitat)
3) Difficulties in adequately predicting the performances (e.g. large uncertainties related to morphological models)
4) Public perception of structures requiring maintenance as having failed.

For any new LCS introduced into the marine environment it will take time for the biological assemblage to reach a diverse community. For mature biological communities to develop, LCS's need to be stable and built in such a way that maintenance will be minimal. Unless LCS's meet these criteria, there is little point in introducing additional features to enhance diversity (for example by enhancing complexity), as attempts to repair the structure will result in considerable degradation of developing communities. As an example many green ephemeral algae are early colonizers and will be abundant on new structures or on structures subject to high disturbance. Designing for low structure damage and avoiding/restricting human access will minimize the growth of green ephemeral algae.

Generally there are differences in the exposure of armour blocks of the various parts of the structure (heads, trunk crest, trunk seaward and leeward sides). However, for preliminary/conceptual design it is recommended to use the same armour size for the whole structure, corresponding to the most exposed part. If the structure is expected to be exposed to oblique wave attack the same rock type should be applied in the whole roundhead. Anyway, for LCS's it is usually chosen to use stones in the trunk and the roundhead of the same size. In this case design can be done according to the new armour stability formulae. As LCS's are low the use fairly gentle slopes does not increase the total required quantity of material significantly. It is therefore recommended to use 1:2 slopes or even gentler slopes. For gentler slopes the structure will be more stable than given by the new formulae. The new formulae are meant to be used for designing typical low- and narrow-crested breakwaters. The crest width should be at least equal to the largest significant wave height, and the crest width should correspond to at least three stones. However, the formulae can also be used for designing wide homogeneous structures, such as reefs, but in that case design by the given formulae will be conservative.

Choosing proper design waves and water levels is especially important for armour layer design of LCS's. The designer must consider the possible increase in wave height due to sealevel rise over the life of the structure. Based on knowledge about waves offshore the wave heights at the location of the LCS to be used in the stability formulae can be estimated by a simple method given in the thesis.

Design of bedding layers, stone filters, and scour protection for LCS's is not treated in detail in this thesis. However as designing these parts is imperative to ensure structural stability the thesis gives descriptions and reference to existing work regarding formulae and further detail. Examination of existing structures suggests that seabed erosion and settlement in some cases present more serious design problems than armour instability.

Recommendations for further research

In Italy long (i.e. large length/gap-ratios) submerging breakwaters exist for which problems with sliding of armour units into the gaps have been monitored. The author believes this is caused by undermining of the toe-berm in the roundheads due to scour. The reason for the failures is most likely due to the effect of low freeboard in combination with long structures which causes large rip-currents in the gaps. This hypothesis should be verified, and possible counter methods to avoid the problem should be developed. For such structures the influence of the rip-currents on the armour and toe stability is also a vital topic for further research.

The present study cannot be used to evaluate the effect of trunk and roundhead slope on the stability. By making wider roundheads or less high roundheads it is possible to use gentler roundhead slopes. Gentler roundhead slopes compared to trunks of LCS's may possibly increase roundhead stability significantly. The influence of structure slope on trunk and roundhead stability is therefore a significant topic for further investigation.

Wave transformation into shallow water and wave breaking characteristics at the LCS is important for the structural stability. The foreshore slope in front of the structure influences the wave breaking and maximum obtainable wave heights. If the foreshore slope is very steep, very large waves which are more damaging to the structure can occur. No specific studies about this subject exist as most existing LCS's are built in areas with gentle foreshore slopes. If the use of LCS's for coastal protection continues increasing in popularity it may be beneficial to perform such a study.

12 Acknowledgements

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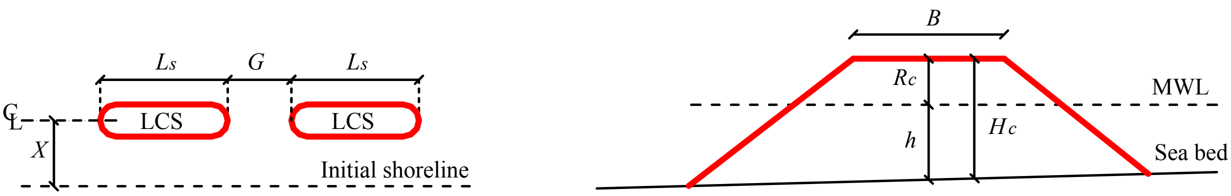
14 Notation



X	Distance between the shore line pre-project and the centre line of the LCS
L_s	Length of the segments at crest level
G	Length of the gaps between the structures at crest level
B	Width of LCS at crest level
R_c	Freeboard, the distance from crest level to MWL (negative if submerged)
h	Water depth at MWL

The height of the structure is: $H_c = h + R_c$.

Figure 14.1. Definition of key parameters related to structural layout of LCS's.

A_e	Cross-section area of erosion, see Chapter 3.2
A_t	Cross-section area of trunk
B	Crest width at crest level, see Figure 14.1 above
D	Damage level “Destruction” (filter layer is exposed to direct wave attack)
$D\%$	Damage parameter, percent erosion of original cross-section area, see Chapter 3.2
D_n	Equivalent diameter of cube $D_n = \sqrt[3]{W_a / \rho_a}$
D_{n50}	Nominal diameter $D_{n50} = \sqrt[3]{W_{50} / \rho_a}$
D_R	Damage parameter, relative displacement within an area, see Chapter 3.2
F_D	Drag force
F_I	Inertia force
F_L	Lift force
g	Constant of gravitational acceleration, $g = 9.82$ m/s^2
G	Gab between breakwater segments, see Figure 14.1 above
h	Water depth at MWL by the structure, see Figure 14.1 above
h_b	Water depth at top of toe berm
h_c	Equilibrium height of a reshaping breakwater
H	Wave height
H_c	Structure height, see Figure 14.1 above
H_m	Mean value of wave heights

H_{m0}	Significant wave height from wave energy spectrum
H_{rms}	Mean value of the root-mean-square wave heights
H_s	Significant wave height, mean value of the 1/3 highest wave heights
H_{tr}	Transition wave height
$H_{x\%}$	Wave height with probability of exceedance x%
ID	Damage level "Initiation of Damage" (a few stones starts to move)
IR	Damage level "Iribarren damage" (big holes in the outer armour layer, but the filter layer is not visible)
K_t	Coefficient for wave transmission
K_r	Coefficient for wave reflection
K	Factor, scalar or constant
L_s	Length of breakwater segments, see Figure 14.1 above
L_p	Wave length at the peak period
L_{0p}	Wave length at the peak period in deep water, $L_{0p} = gT_p^2 / 2\pi$
m	Beach foreshore slope, $m = 0.02$ if the foreshore slope is 1:50
n	Porosity, $n = V_v/V$ or equally $n = (V - W_s/\rho_s)/V$
N	Number of displaced stones
N_a	Damage parameter, the total number of units within a strip of horizontal width D_n, see Chapter 3.2
ND	Damage level "No damage" (maybe one or two loose stones starts rotating)
N_{od}	Damage parameter, number of displaced units within a strip with width D_n, see Chapter 3.2
N_s	Stability number, $N_s = H/(\Delta D_n)$, where H could be e.g. H_s and D_n is usually D_{n50}
N_s^*	Spectral stability number by Ahrens (1987), $N_s^* = \frac{H_s}{\Delta D_{n50}} s_{0p}^{-1/3}$
N_{ta}	Total number of units in armour layer
N_z	Number of waves
P	Notional permeability by Van der Meer (1990). $P = 0.4$ for three layer a conventional breakwater, $P = 0.5$ for a two layer structure, and $P = 0.6$ for a homogeneous structure
R	Mean of the head radii. Parameter for damage calculation of S_{head}
R_c	Crest freeboard, positive when emerged, see Figure 14.1 above
s_m	Wave steepness parameter $s_m = 2\pi H_s / gT_{0m}^2$. Based on deep water wave period T_{0m}
s_p	Peak wave steepness at a given water depth

s_{0p}	Wave steepness $s_{0p} = H_s/L_{0p}$. Based on deep water wave length L_{0p}.
S	Damage parameter, see Chapter 3.2
S_{head}	Damage parameter for roundhead, method by Vidal *et al.* (1995), see Chapter 3.4
S_u	Set-up (increase in water level leeward of the structure)
S_θ	Parameter for angular spreading of waves according to the cosine power spreading function, see Mitsuyasu *et al.* (1975)
T_m	mean wave period
T_p	Peak wave period (mean period of the highest 1/3 of the waves)
V	Total volume (voids and stone)
V_f	Characteristic flow velocity
V_v	Volume of voids
V_e	Eroded volume corresponding to eroded area A_e
W_s	Weight of stones
W_a	Armour unit weight
W_{50}	Median armour unit weight derived from the mass destribution curve
X	Distance between LCS and the shore line, see Figure 14.1 above
Y	Width of trunk test section for damage measurements
α	Structure slope angle to horizontal
β	Angle of wave attack to breakwater alignment (zero for head on wave attack, i.e. wave fronts parallel to breakwater)
γ	$\gamma = H_s/h$. Factor describing depth limitation of waves, the factor is depending on the foreshore slope and wave steepness
θ	Angle of roundhead sector for calculation of damage parameter S_{head}
Δ	Relative density of material: $\Delta = \rho_a / \rho_w - 1$
ρ_s	Mass density of stones
ρ_a	Mass density of armour units
ρ_w	Mass density of water
ν	Kinematic viscosity, $\nu \approx 10^{-6}$ m^2/s
ξ_m	Surf similarity parameter based on wave steepness s_m

Appendix A Inventory of European LCS's

The appendix summarizes statistics on LCS's geometry mainly within EU. The information is collected and analyzed by the author for DELOS WP1.1 *"Inventory of engineering properties of LCS"*, and the text has earlier partly been published through the DELOS project as part of the Work Package 1.1, Delivery 05, see Kramer (2001 and 2002). At the Internet www.delos.unibo.it all collected data and documents produced within WP 1.1 can be downloaded. In the Microsoft Excel Workbook "LCS_Inventory_Statistics.xls" parameters and calculations used in this document can be found.

A database is assembled from 175 completed questionnaires; 150 are about schemes within EU, 24 from USA and 1 from Japan. The database contains only geometrical information due to the fact that too limited information has been available about e.g. morphological changes and hydrodynamic conditions.

In Japan a statistical analysis of detached breakwaters exists, see Takaaki (1988). The information about the breakwater projects in USA is mostly based on structures in the report by Chasten *et al.* (1993). Information from these references is used for comparisons.

The inventory was established in the following way:

1) A brief description was given for each LCS (Appendix A.1). The description was given for all kinds of LCS for as many structures as possible in each country.
2) Some structures/locations were selected for further investigations
3) A more detailed description was given for the selected structures/locations (Appendix A.2). This part mainly focuses on shore parallel structures including shore-attached structures, which are perpendicular to shoreline if part of the scheme.
4) Analysis and statistics were performed on the collected data. Some of this information is included in Appendix A.3 and some information about country-specific geometry is given in Appendix A.4.

Some minor changes to the following introductory chapters have been made in order to avoid repetitions.

A.1 Questionnaire for inventory on LCS's, brief description

According to DELOS WP 1.1, an inventory for existing low crested structures (LCS) must be established. As low crested structure we mean structures designed to be submerged or regularly overtopped by waves. The brief descriptions will be presented on www.delos.dk and the structures to be selected for further investigations will be discussed.

How to use this document

In this document, you must give a brief description of a specific LCS. This description must be no more than one or two A4-pages and information can be given approximately. The description must be completed within this digital document. When completed, please attach the document to an email and send it to i5mkr@civil.auc.dk.

The filenames for the documents must include the participant code, the Country Code (as used on the Internet for Country Code Domains) and a Location-number between 001 and 999. The filenames for UB collecting information from East Italy (see special Country Code below) will therefore be "UB_EIT_001.doc" till "UB_EIT_999.doc". Each participant must provide a map of the country showing all the locations of the sites of interest, the Location-numbers must appear on this map.

Inputs come from:
UPC: Spain (Country Code ES)
DHI: Denmark (Country Code DK)
MOD & UR3: West Italy (Country Code WIT)
UB: East Italy (Country Code EIT)
AUTH: Greece (Country Code GR)
INF: Holland (Country Code NL)
UCA: non European LCS by literature study (Country Code nonEU)
UoS: U.K. (Country Code UK)

Subsequent information should be given for each subject (Bold type). Below each heading, the non-bold texts are given as example.

Location

Please give a short description and if possible insert a bit of topographic atlas e.g. on a scale of 1:25,000.

Main motive for building the LCS

E.g. coast erosion.

Impacts on bio-environment

E.g. wildlife has increased a lot after the LCS was built.

Socio-economic impact

E.g. recreational activities have increased a lot after the LCS was built.

System Layout (dimensioned sketch)

As a minimum the distance form the shoreline, the width and the length of the structures should appear.

Typical cross section (dimensioned sketch)

As a minimum the approximately freeboard by MWL (R_C), the width, the height and the water depth should be given.

Indication of water level variations

E.g. in the area there is no tide of concern, or usually the tide in the area varies ±2 meters.

Existence of detailed information

E.g. a lot of detailed information can be given by request, or it is hardly impossible to obtain more information. The location is/is not appropriate for further investigation.

A.2 Questionnaire for inventory on LCS's, detailed description

The detailed inventory (described below) concerns shore parallel structures including shore-attached structures, which are perpendicular to shoreline if part of the scheme. This inventory will be established through a digital questionnaire located at www.delos.dk.

How to use this document

In this document, you can give a detailed description of a specific LCS. The description must be completed within this digital document. Just type the text in the tables, insert relevant pictures, drawings, sketches etc. and save the document. Only relevant information should be included in the document; existing non-used tables, sketches etc. present in this document must be deleted. The existing figures etc. are meant to be guidelines that can be changed for a specific environment. But please keep the structure of the document intact.

When completed, please attach the document to an email and send it to i5mkr@civil.auc.dk.

The filenames for the documents must include the participant code, the Country Code (as used on the Internet for Country Code Domains) and a Location-number between 001 and 999. It is very important that the same Location-number is used as for the brief description. The letters "det" must also be included to indicate that the detailed version of the questionnaire is used. The filenames for UB collecting information from East Italy (see special Country Code below) will therefore be "UB_EIT_det_001.doc" till "UB_EIT_det_999.doc". Each participant must provide a map of the country showing all the locations of the sites of interest, the Location-numbers must appear on this map.

A.2.1 Formalities

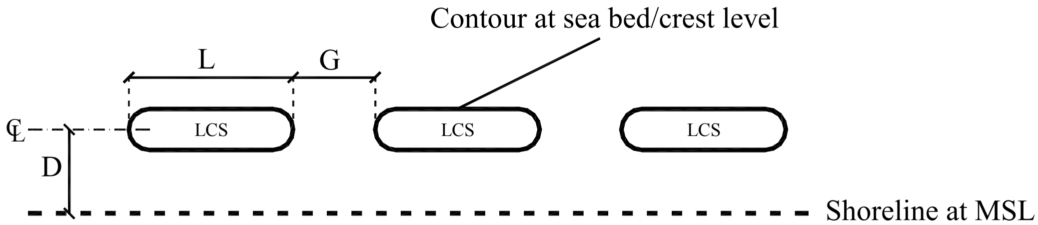
Participant code and who to contact.	
E-mail	
This date (today, mm:dd:yyyy) and revision number (A..Z).	
Location of LCS.	
Start date, length and/or end of works. Have there been any later changes? If so, when?	
Design life - the minimum length of time the beach management scheme is designed to last.	
Which tools and regulations are used for the design formulae (mathematical models, model tests, engineering experience, standards, and recommendations).	
Who fund the work (e.g. Public Administration or private company)?	
Costs.	

A.2.2 Geometry and construction materials

System layout (aerial view)

Are shore attaching structures present (e.g. groins)?	☐Yes ☐No
Are emerging head islands present?	☐Yes ☐No

The following sketch concerns only shore parallel LCS; if the layout is different you must insert another sketch and specify parameters like the ones suggested. If a picture is available please insert it too.



The typical layout is given at Sea Bed (index SB) and at Crest Level (index CL).

Parameter	Description	Fill in box	unit
D	Distance from shoreline		meters
L_{SB}	Length of LCS at sea bed		meters
L_{CL}	Length of LCS at crest level		meters
G_{SB}	Gap between LCS at sea bed		meters
G_{CL}	Gap between LCS at crest level		meters
n	Number of LCS in system		

Remarks

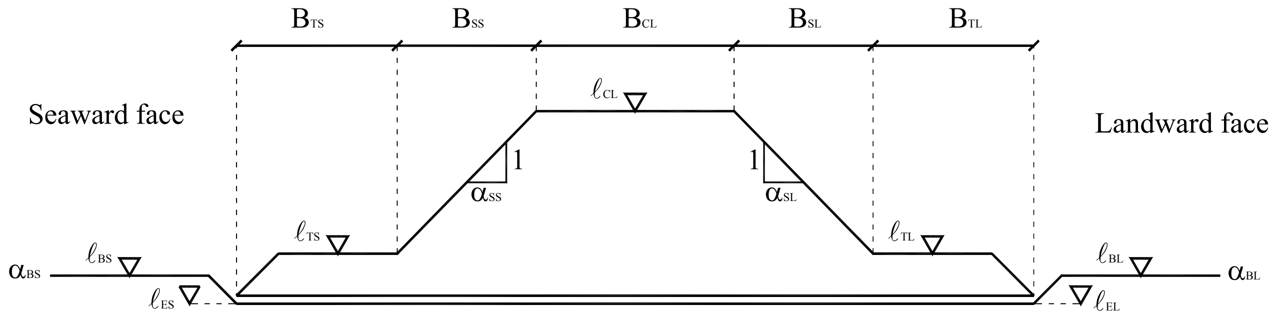
Bathymetry of sea bed and beach

Please insert a dimensioned sketch if possible.

Description of bathymetry when LCS were build Is detailed information (measurements) available? If so, please explain.

Trunk cross section/contour geometry – outer profile

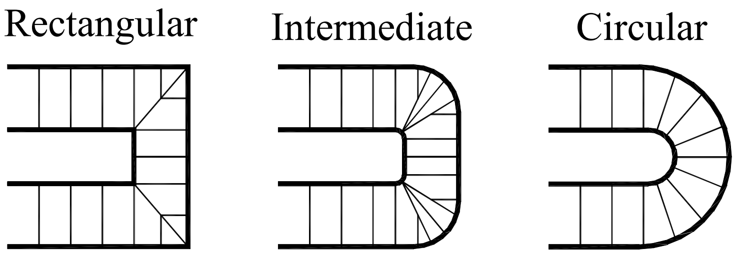
If shore attached structures perpendicular to shoreline are present, please insert a sketch with typical longitudinal section and typical selected cross sections. Specify parameters as the ones given below. If the layout does not fit the following sketch please insert another sketch.



Parameter	Description	Fill in box	unit
α_{BS}	Steepness of sea bed, seaward		
α_{BL}	Steepness of sea bed, landward		
α_{SS}	Steepness of slope, seaward		
α_{SL}	Steepness of slope, landward		
ℓ_{BS}	Level of sea bed at seaward toe		meters
ℓ_{ES}	Level of excavation, seaward		meters
ℓ_{TS}	Level of toe, seaward		meters
ℓ_{CL}	Level of crest		meters
ℓ_{BS}	Level of sea bed at landward toe		meters
ℓ_{ES}	Level of excavation, landward		meters
ℓ_{TS}	Level of toe, landward		meters
B_{TS}	Width of toe, seaward		meters
B_{SS}	Width of slope, seaward		meters
B_{CL}	Width of crest		meters
B_{SL}	Width of slope, landward		meters
B_{TL}	Width of toe, landward		meters

Remarks (e.g. different layout along shoreline, other important parameters).

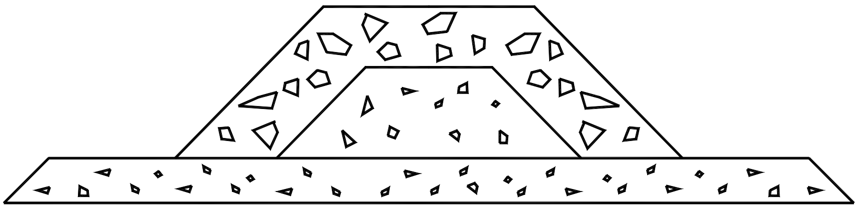
Round head contour geometry



What is the shape of the round head?
☐ Rectangular
☐ Intermediate
☐ Circular

Description of layers

Please insert a dimensioned sketch with the typical cross-section composition.



For each layer, please provide the following information.

Layer type e.g. ARMOUR LAYER CHARACTERISTICS			
Parameter	**Description**	**Fill in box**	**unit**
	Material (e.g. quartzite, concrete)		
	Shape of blocks (e.g. quarry rock, sea stones, cubes)		
ρ_r	Mass density of material		kg/m^3
D_{n50}	Nominal diameter		meters
Gr	Grading of the material (D_{85}/D_{15})		
	Geotextile between layers?	☐Yes ☐ No	

Remarks (e.g. details on geotextile)

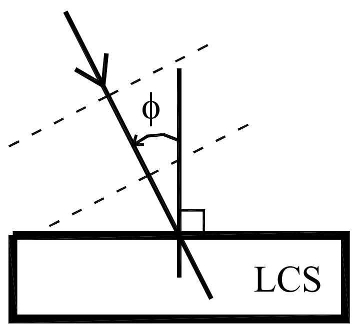
Construction method

How have the stones been placed?
☐ Dumped with barges
☐ Placed with barges
☐ Land based operation
☐ Other:

Sequence of operation.
☐ Construction started upstream
☐ Construction started downstream

A.2.3 Local meteomarine conditions at the structure

Waves



Parameter	Description	Fill in box	unit
H_S	Design significant wave height		meters
T_P	Design peak period		seconds
ϕ	Design wave incidence angle		degree

Remarks (provide information on wave statistics and wave spectra if available, e.g. H_S corresponding to return periods 1 month, 1 y, 10 y, 50 y. Please specify the source of the data)

Water levels

TIDAL WATER LEVEL VARIATIONS			
Parameter	**Description**	**Fill in box**	**unit**
HAT	Highest astronomical tide level		meters
MHWL	Mean tide high water level		meters
MWL	Mean water level		meters
MLWL	Mean tide low water level		meters
LAT	Lowest astronomical tide level		meters

Water level statistics (If available, please provide information on design water level and tide and surge generated water levels corresponding to return periods 1 month, 1 y, 10 y, 50 y)

Current

Tidal currents

Description & statistics if available

Surge generated currents

Description & statistics if available (e.g. mean velocities as function of water depth/distance to shore line)

A.2.4 Sea bed and beach characteristics, incl. sediment transport

Description of the coast (e.g. bar type coast with gentle slope or plane coast with steep slope)

Natural sea bed material at surface

Parameter	**Description of sea bed material**	Fill in box	unit
	Material (e.g. quartzite)		
ρ_r	Mass density of material		kg/m^3
D_{n50}	Nominal diameter grain size		meters
Gr	Grading of the material (D_{85}/D_{15})		

Remarks (provide grain distribution if available)

Natural beach material at surface

Parameter	Description of beach material	Fill in box	unit
	Material (e.g. quartzite)		
ρ_r	Mass density of material		kg/m^3
D_{n50}	Nominal diameter grain size		meters
Gr	Grading of the material (D_{85}/D_{15})		

Natural supply?	☐ Yes ☐ No
Supplied by beach nourishment?	☐ Yes ☐ No

Remarks (provide grain distribution if available)

Artificial beach nourishment

Description of nourishment

Parameter	Description of artificial nourishment	Fill in box	unit
	Material (e.g. quartzite)		
ρ_r	Mass density of material		kg/m^3
D_{n50}	Nominal diameter		meters
Gr	Grading of the material (D_{85}/D_{15})		

Remarks (provide grain distribution if available)

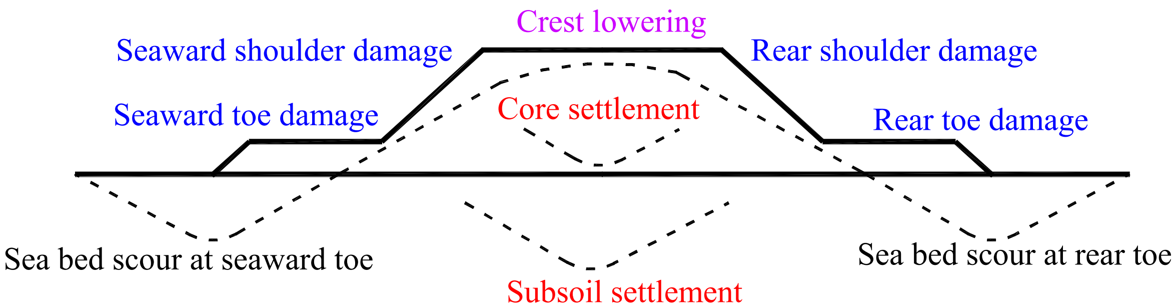
Sediment transport

Description of the sediment transport (e.g. direction and amount of transport, distribution over the coastal profile)

Parameter	Description of sediment	Fill in box	unit
	Material (e.g. quartzite)		
ρ_r	Mass density of material		kg/m^3
D_{n50}	Nominal diameter		meters
Gr	Grading of the material (D_{85}/D_{15})		

A.2.5 Structural performance

Definition of failure modes



Please insert a sketch with dimensions of LCS cross-section when it was build compared to the appearance now (like the figure of failure modes) if possible.

In the following please specify damages by failure mode (see figure of failure mode definition) and amount of damage. If you know the reason for the problems/failures (e.g. extreme wave climate/water level), please type it in the description boxes.

Materials

Problems caused by deterioration?	☐ Yes ☐ No
Problems caused by breakage?	☐ Yes ☐ No

Description of the condition of the materials

Settlement of the structure

Description of settlements of core/subsoil (e.g. instabilities in foundation, internal erosion). Please specify settlement in meters.

Local erosion of sea bed/scour

Description of erosion/scour by roundheads (please specify scour depth)

Description of erosion/scour by trunk (please specify scour depth)

Erosion and instability of slopes, shoulders, crest and toes

Stage of damage
☐ No or marginal damage
☐ Moderate to severe damage
☐ Failure

Description of displacements of structural material (provide sketch if possible)

Damage parameters

The definition of a displaced unit is, when a unit is displaced by more than D_{n50}. Try to give an estimate of the following damage parameters relevant to armour.

Parameter	Description	Fill in box	unit
The relative number of displaced units	$D(\%) = \frac{n_d\,(\text{number of displaced units})}{\text{Total number of units}} \cdot 100$		%
The strip displacement	$N_{od} = \frac{n_d}{L / D_{n50}}$, L is the length of LCS		

A.2.6 Socio-economic aspects

What regime of property has the coast at this site?

Private ☐, Public full free access ☐, Public limited access ☐, Natural reserve ☐, Don't know ☐,
Other (please specify):

Who decided that an LCS should be built at that site?

Individual, acting for private purpose ☐
Individual, acting for public purpose (e.g. Natural park administrator) ☐
Local authority (e.g. city council) ☐
Regional authority (e.g. province level) ☐
National authority (e.g. ministry) ☐
Don't know ☐
Please give name of the authority whenever applicable:

What was the main motive for building the LCS?

Coast erosion ☐
Inducing or maintaining recreational activity ☐, please specify:
Environmental concern ☐, please specify:
Other ☐, please specify:
Don't know ☐

Was that LCS part of a larger coastal management plan?

Yes ☐, please specify:

No ☐, please specify:

Don't know ☐

Public opinion on that LCS:
Construction was accompanied by public protest ☐
The public did not react ☐
Public opinion asked for the LCS ☐
Local commerce asked for the LCS ☐
Don't know ☐
Other (please specify):

Description of the coast:
Urban ☐, Densely constructed ☐, Scarcely constructed ☐, No apparent construction ☐
Are there dunes? Yes ☐, No ☐
Has commercial activity changed significantly after construction of the LCS?
hotels construction: More hotels ☐, Less hotels ☐, Unaffected ☐, Don't know ☐
bars and similar construction: More ☐, Less ☐, Unaffected ☐, Don't know ☐
advertising for the area: More ☐, Less ☐, Unaffected ☐, Don't know ☐
other (specify):

Visual impact of LCS not already described in Part B: Are there parts of the LCS visible under average conditions? Poles ☐, Cables ☐, Reefs ☐,
Others (please specify):

Water quality changes since LCS construction
Are there episodes of water turbidity since construction?

No ☐, Rare ☐, Often ☐, Permanent ☐

Were there episodes of water turbidity before construction?

No ☐, Rare ☐, Often ☐, Permanent ☐

Has water quality otherwise been affected (for example, more or less detritus accumulating)? Please describe:

How would you qualify the following recreational activities at or around the LCS? (DK = Don't know)

Fishing (recreational)	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Seafood collecting	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Wildlife watching	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Sunbathing and similar	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Scuba diving	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Sailing and similar	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Other (specify)	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐

Could you describe those recreational activities before the LCS was built? (DK = Don't know)

Fishing (recreational)	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Seafood collecting	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Wildlife watching	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Sunbathing and similar	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Scuba diving	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Sailing and similar	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐
Other (specify)	Intense ☐	Moderate ☐	Scarce ☐	Absent ☐	DK ☐

Has that LCS had an environmental impact assessment before being built? Yes ☐, No ☐, Don't know ☐
Could you give its references and location (specify)?

Has there been an economic study on that LCS,
before it was built? Yes ☐, No ☐, Don't know ☐, References:
after it was built? Yes ☐, No ☐, Don't know ☐, References:

A.2.7 Ecological aspects

What are the dominant species on the structures?

What are the dominant species in the sediment and fish assemblages around the structures?

Were any environmental changes observed following the construction of the structure (e.g. increase of water turbidity, floating algal debris)?

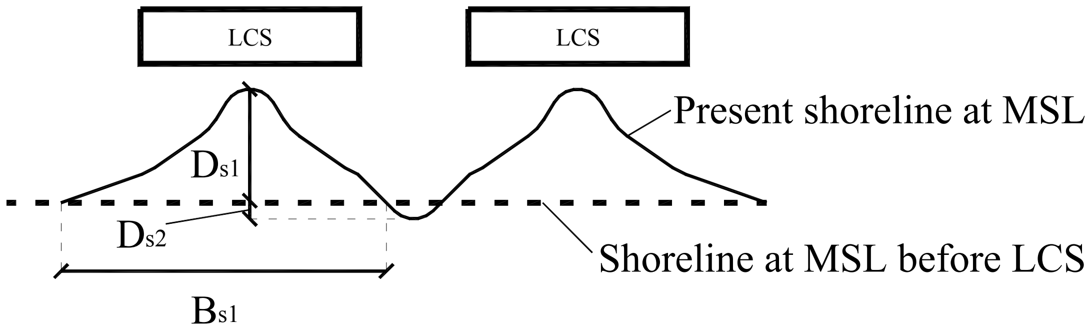
A.2.8 Coastal protection performance

Bathymetry and beach evolution

Description of historical beach evolution before LCS was built (10-20 years).

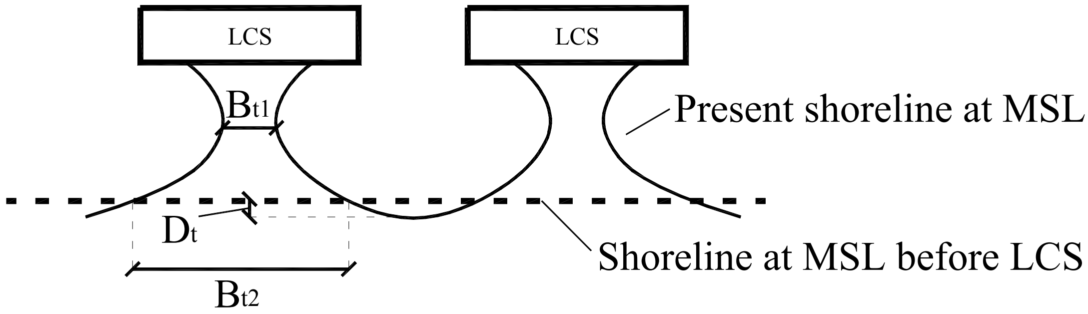
Description of beach evolution after LCS was built up to now.

Salient formation



Parameter	Description	Fill in box	unit
D_{s1}	Max distance between new and old shoreline, seaward		meters
D_{s2}	Max distance between new and old shoreline, landward		meters
B_{s1}	Width of salient at old MSL		meters

Tombolo formation



Parameter	Description	Fill in box	unit
D_t	Distance between new and old shoreline, landward		meters
B_{t1}	Minimal width of tombolo		meters
B_{t2}	Width of tombolo at old MSL		meters

Renourishment

Description of renourishment (add more fill) (e.g. amount, how often)

Down drift erosion

Please insert a sketch if relevant.

Description of down drift erosion (morphological impact, e.g. down drift erosion length and maximal down drift shoreline retreat)

A.2.9 Problems in general

Description of other problems/impacts

A.3 Types and geometry of LCS's in the inventory

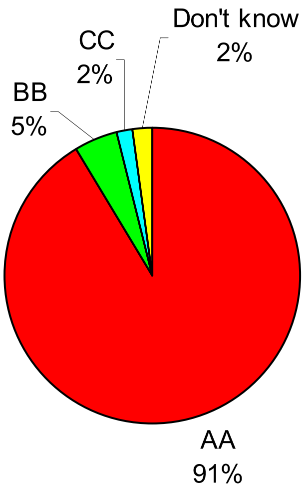
Each completed questionnaire is denoted a *scheme*. All schemes have been categorized with respect to *purpose* of construction and *type* of structures. In total 175 schemes have been investigated.

Main purpose

AA) Beach and land protection against erosion
BB) Coastal protection for ecological reasons
CC) Protection of harbours, inlets, outlets, channels etc

Table A.1. Main purpose of the investigated LCS's. Contents are in number of schemes.

Country	AA	BB	CC	Don't know	Total
DK	4	0	0	0	4
NL	2	2	2	0	6
IT (UR3/MOD)	18	0	0	0	18
IT (UB)	56	0	0	0	56
GR	4	0	0	0	4
ES	28	0	0	0	28
UK	27	2	1	4	34
US	20	4	0	0	24
Japan	1	0	0	0	1
Total	160	8	3	4	175

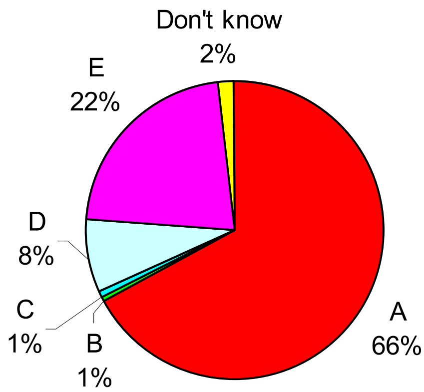


Construction types

A) Detached LCS
B) T-shaped LCS
C) L-shaped LCS
D) Groins
E) Combinations/other

Table A.2. Construction types. Contents are in number of schemes.

Country	A	B	C	D	E	Don't know	Total
DK	2	1	0	0	1	0	4
NL	0	0	0	1	5	0	6
IT (UR3/MOD)	10	0	0	0	8	0	18
IT (UB)	49	0	0	0	7	0	56
GR	4	0	0	0	0	0	4
ES	18	0	0	0	10	0	28
UK	11	0	1	13	6	3	34
US	22	0	0	0	2	0	24
Japan	1	0	0	0	0	0	1
Total	117	1	1	14	39	3	175



In Table A.1 it is seen that most structures are built for beach protection against erosion (AA). It is also clear from Table A.2 that most of the investigated schemes contain only detached LCS's.

A.3.1 Structural layout in selected schemes

The LCS's in the Netherlands are rather atypical; only one scheme is actually for beach protection. The structural parameters are very diffuse and are not interesting for comparison with other layouts. Therefore the schemes in NL are not included in the statistics.

Many schemes in UK have been excluded due to the fact that too limited information is available, or only simple groins are present.

Almost all of the remaining schemes contain segmented detached breakwaters. Therefore all schemes with detached breakwaters are included in the subsequent statistics.

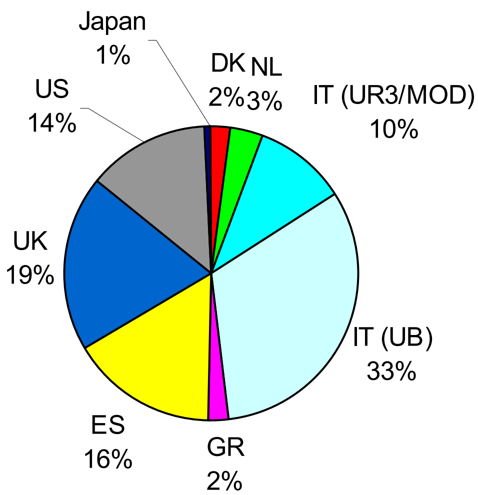
Some schemes contain different types of structures e.g. both detached breakwaters and groynes. Only the detached breakwaters are included in the statistics. The detached breakwaters are grouped in *cases* defined as structures with almost the same main structure geometry and lay-out-geometry (geometrical parameters do not differ more than app. 10%). In total the presented statistics is based on 185 cases containing 1483 structures. The parameters for these structures can be found subsequent in Table A.8.

A.3.2 Locational distribution of collected information

The following shows in which countries the schemes are located.

Table A.3. Locational distribution of information.

Country	Partner	Number of schemes	Number of selected cases	Structures in selected cases
DK	DHI	4	3	40
NL	INF	6	0	0
IT	UR3/MOD	18	25	62
IT	UB	56	83	1038
GR	AUTH	4	4	11
ES	UPC	28	37	66
UK	UoS	34	9	31
US	UCA	24	24	235
Japan	UCA	1	0	0
Total		175	185	1483



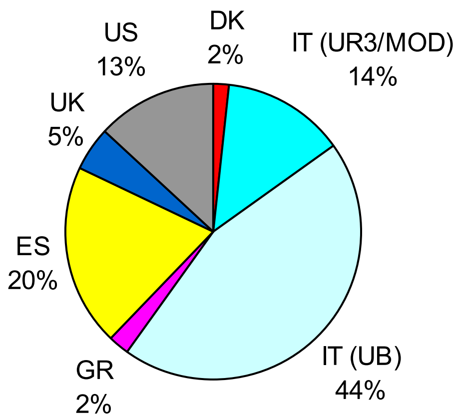


Figure A.1. Left: Distribution of schemes. Right: Distribution of selected cases.

It is clear that the main part (78%) of the LCS's are located in Spain and Italy.

A.3.3 General statistics in selected cases

The selected cases are categorised in EU cases and US cases. Parameters for the selected cases can be found in Table A.8. In the following tables two average values are calculated:

1) Average value of the specified parameter in the selected cases
2) Average value of the specified parameter for all considered structures

Table A.4 General statistics, EU cases. Total no. Structures: 1248. Total no. of cases: 161

General statistics, EU cases						
Parameter	**Cases**	**Structures**	**Min**	**Average (case)**	**Average (structure)**	**Max**
Length L_s	159	1245	25	270.21	125.82	3000
Gap G	84	1102	10	40.64	38.30	300
Distance D	157	1243	15	120.89	120.29	350
Freeboard R_c	147	1098	-3	0.17	0.80	3.6
Width B	140	1076	2	7.48	5.05	25
Depth h	136	1018	1	3.74	3.05	8.5
Tidal range	159	1243	0	0.77	0.43	10

Table A.5 General statistics, EU & US cases. Total no. Structures: 1483. Total no. of cases: 185.

General statistics, EU & US cases						
Parameter	**Cases**	**Structures**	**Min**	**Average (case)**	**Average (structure)**	**Max**
Length L_s	183	1480	15	241.97	113.18	3000
Gap G	105	1334	10	42.95	44.44	300
Distance D	175	1433	15	120.42	118.75	370
Freeboard R_c	161	1299	-3	0.26	0.83	3.6
Width B	149	1200	2	7.45	5.38	25
Depth h	156	1237	0.45	3.48	2.79	8.5
Tidal range	175	1399	0	0.80	0.49	10

A.3.4 Distribution of structural ratios

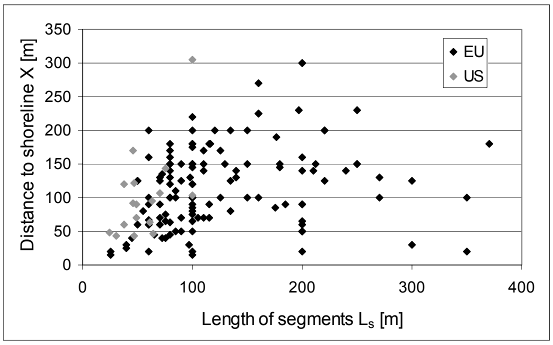
7 different structural ratios have been investigated as indicated in Table A.6 and Table A.7.

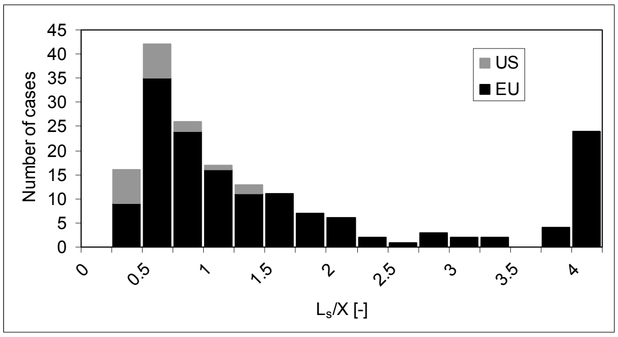
Table A.6. Structural ratios, EU cases. Total no. Structures: 1248. Total no. of cases: 161.

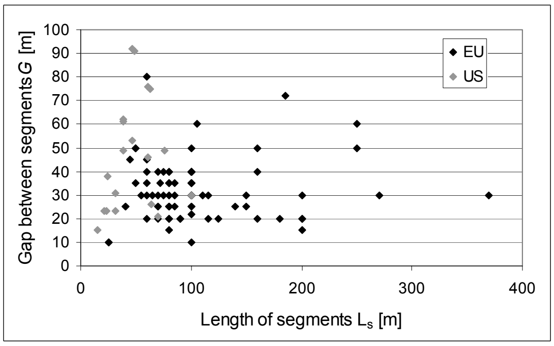
Structural ratios, EU cases						
Ratio	**Cases**	**Structures**	**Min**	**Average (case)**	**Average (structure)**	**Max**
L_s/D	157	1243	0.30	2.99	1.39	46.67
L_s/G	84	1102	0.20	3.74	3.09	13.33
R_c/B	140	1076	-0.83	0.06	0.25	0.65
H_c/B	127	976	0.06	0.71	0.95	2.25
R_c/h	134	998	-0.80	0.10	0.33	2.00
B/h	127	976	0.55	2.12	1.73	8.33
R_c/tr	91	470	-10.00	-0.30	1.07	5.00

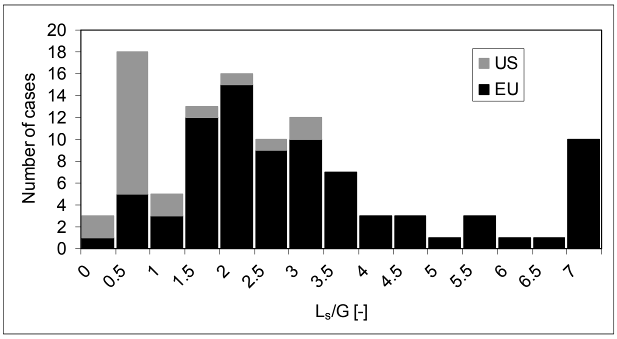
Table A.7 Structural ratios, EU & US cases. Total no. Structures: 1494. Total no. of cases: 186

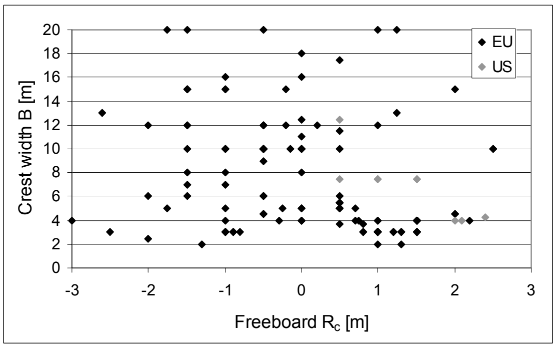
Structural ratios, EU & US cases						
Ratio	**Cases**	**Structures**	**Min**	**Average (case)**	**Average (structure)**	**Max**
L_s/D	176	1444	0.27	2.75	1.27	46.67
L_s/G	106	1345	0.20	3.22	2.70	13.33
R_c/B	150	1211	-0.83	0.08	0.24	0.65
H_c/B	135	1109	0.06	0.71	0.89	2.25
R_c/h	146	1197	-0.80	0.15	0.39	2.00
B/h	135	1109	0.55	2.35	2.58	27.78
R_c/tr	100	609	-10.00	-0.15	1.14	6.58

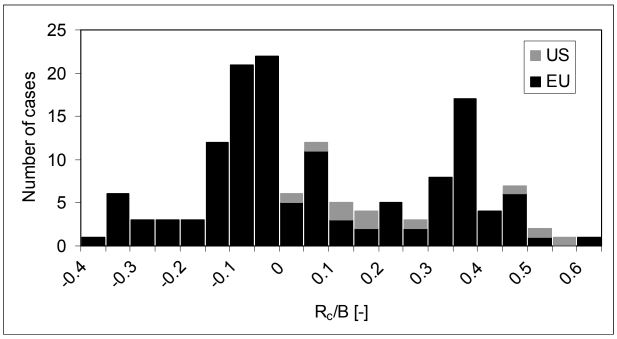


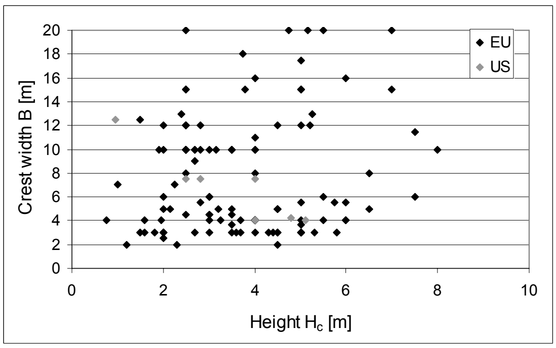


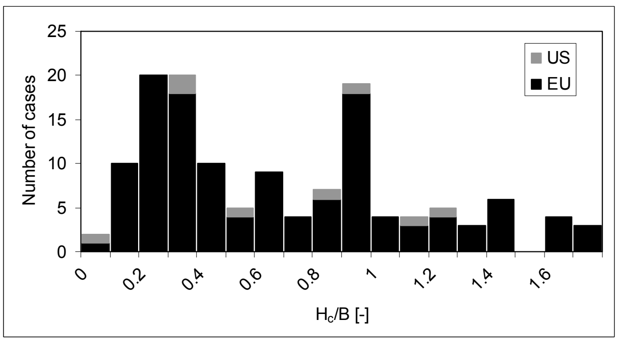












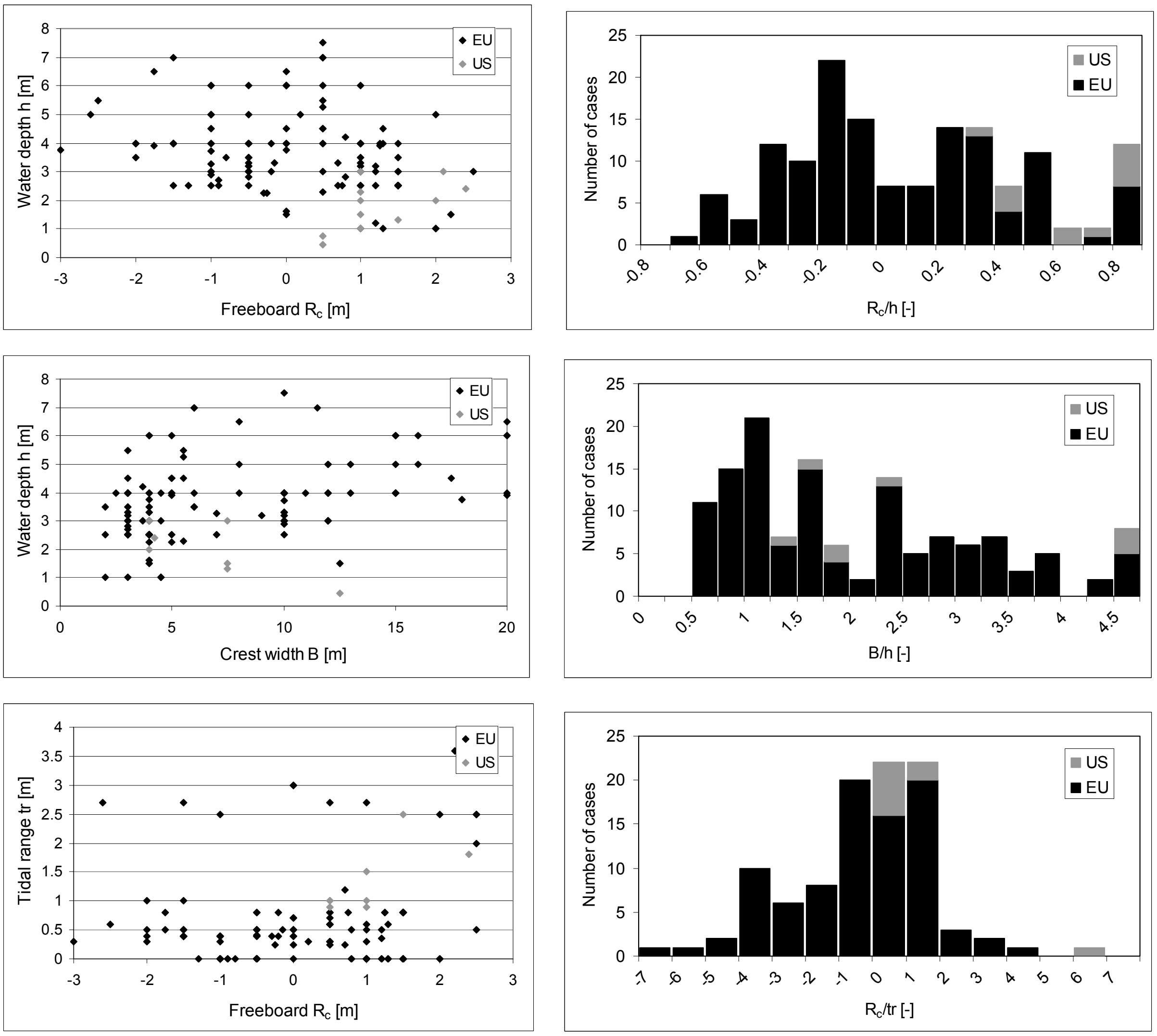


Figure A.2 Distribution of structural ratios

The R_c/h ratio was expected to depend on the range of the tide. Therefore the cases have been divided in low-tide cases and in high tide cases. Definition of low tide: Astronomic range of the tide <1m (117 cases). Definition of high tide: Tidal range ≥ 1m (21 cases).

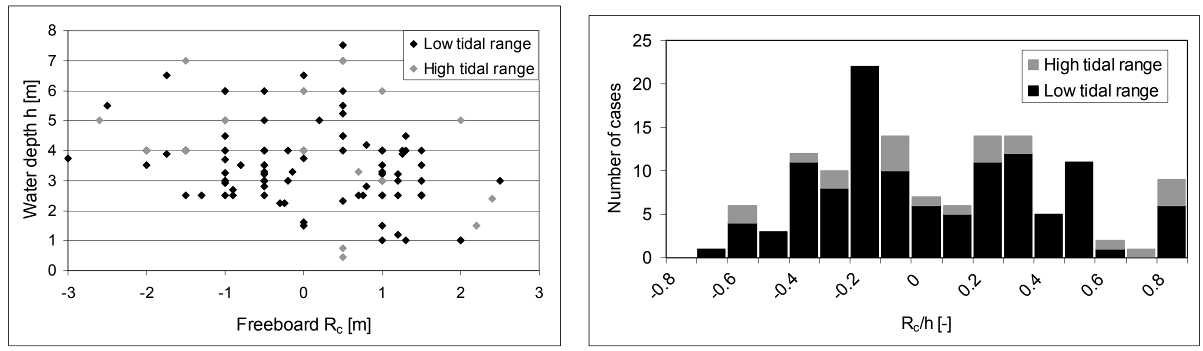


Figure A.3 Distribution of height-ratio in cases of high and low tidal range

In order to find possible relations between the investigated structural ratios (2002) produced scatter diagrams with many combinations of structural ratios. Kramer (2002) concluded that generally very weak correlations between the ratios of the structural parameters were present in the cases.

Table A.8. Inventory: Geometrical parameters in the selected cases.

Country	Filename	Name of site	Purpose	Type	N_{umber}	Length L_s [m]	Gap G [m]	Dist. X [m]	Freeboard R_c [m]	Width B [m]	Depth h [m]	Tide tr [m]
DK	DHI_DK_002	Skagen	AA/CC	E	24	40	25	25	1	3	1	0.3
DK	DHI_DK_003	Lønstrup	AA	A	10	45	45	40	1.3	2	1	0.6
DK	DHI_DK_004	Liseleje	AA	A	6	60	300	60	1.2		1.2	0.36
IT	UR3_MOD_EIT_01_01	Pellestrina	AA	E	8	200		300	-1.5	8	4	1
IT	-		AA	E	8	200		300	-2	6	4	1
IT	UR3_MOD_EIT_04_01	Silvi Marina	AA	A	6	160	20	225	0	10	4	0.5
IT	-		AA	A	3	160	50	225	-0.5	10	4	0.5
IT	UR3_MOD_EIT_04_02	Pescara Sud	AA	A	1	200		90	0	10		0.5
IT	UR3_MOD_EIT_04_03	Casalbordino	AA	E	1	1000		350	0	10	4	0.4
IT	-		AA	E	1	700		350	-1	10	4	0.4
IT	-		AA	E	1	1000		350	-1.5	10	4	0.4
IT	UR3_MOD_WIT_08_01	Guardia Piemo	AA	A	1	490		60	-1.5	15	4	0.5
IT	UR3_MOD_WIT_08_02	Diamante	AA	A	1	200			-1.5	15		0.4
IT	UR3_MOD_WIT_08_03	Paola	AA	A	1	500		60	-0.5	12	5	0.5
IT	UR3_MOD_WIT_08_04	Paola-San Luci	AA	E	17	500		55	-1.75	20	6.5	0.5
IT	UR3_MOD_WIT_08_05	Briatico	AA	A	1	530		80	-1.5	12	4	0.4
IT	UR3_MOD_WIT_08_06	Montebello Joni	AA	E	1	300		30	-1.5	12		0.4
IT	UR3_MOD_WIT_08_07	Amendolara	AA	A	1	700		80	-0.5	20	6	0.5
IT	UR3_MOD_WIT_09_01	Castel Volturno	AA	E	1	1700		80	-0.5	12	3	0.4
IT	UR3_MOD_WIT_10_01	Fiumicino-Foce	AA	E	1	1400		200	-2	25	3.5	0.4
IT	UR3_MOD_WIT_10_02	Ostia (I)	AA	E	1	3000		150	-1.5	15	4	0.4
IT	UR3_MOD_WIT_10_03	Ostia (II)	AA	E	1	2000		95	-1.5	20	4	0.5
IT	-		AA	E	1	2000		95	-2	12	4	0.5
IT	UR3_MOD_WIT_10_04	Nettuno	AA	A	1	700	90	150	-0.5	6	3.5	0.4
IT	-		AA	A	1	800	90	200	-0.5	6	3.5	0.41
IT	UR3_MOD_WIT_13_01	Golfo di Patti	AA	A	1	350		100	-1	8	5	0.4
IT	UR3_MOD_WIT_13_02	Agrigento	AA	A	1	270	30	100	-1	15	6	0.4
IT	-		AA	A	1	85	30	100	-1	15	6	0.4
IT	UB_EIT_2_1	From porto Gari	AA	A	60	100	40	150				0.8
IT	UB_EIT_2_2	Casal Borsetti	AA	A	10	100	40	150	1.5	4	2.5	0.8
IT	UB_EIT_2_3	Punta Marina	AA	E	1	3000		300	-0.2	12	3	0.8
IT	UB_EIT_2_4	Lido Adriano	AA	A	17	100	40	180	1.5	4	2.5	0.8

Country	Filename	Name of site	Purpose	Type	N_{umber}	Length L_s [m]	Gap G [m]	Dist. X [m]	Freeboard R_c [m]	Width B [m]	Depth h [m]	Tide tr [m]
IT	UB_EIT_2_5	Lido di Dante	AA	A	2	370	30	180	-0.5	12	3	0.8
IT	UB_EIT_2_6	Lido di Savio-Li	AA	E	25	100	50	100	1.5	4	2.5	0.8
IT	UB_EIT_2_7	Cesenatico	AA	A	30	100	40	220	1.5	4	2.5	0.8
IT	UB_EIT_2_8	S. Mauro-Gatte	AA	A	14	100	22	175	1.5	4	2.5	0.8
IT	UB_EIT_2_9	Bellaria-Igea M	AA	A	27	100	25	100	1.5	4	2.5	0.8
IT	-		AA	A	27	100	25	200	1.5	4	2.5	0.8
IT	UB_EIT_2_10	Rimini	AA	A	55	100	35	150	1.5	4	2.5	0.8
IT	UB_EIT_2_11	Misano	AA	A	7	100	30	100	1.5	4	2.5	0.8
IT	UB_EIT_2_12	Cattolica	AA	A	18	100	30	75	1.5	4	2.5	0.8
IT	UB_EIT_3_1	Gabicce	AA	A	21	80	20	100	0.75	4	2.5	0.8
IT	UB_EIT_3_2	Gabicce	AA	A	1	700		15	-0.5			0.8
IT	UB_EIT_3_3	Casteldimezzo	AA	A	1	350		20	1	3		0.6
IT	UB_EIT_3_4	Fiorenzuola di	AA	A	9	115	20	180.00	-1	10	2.9	0
IT	UB_EIT_3_5	San Marino di P	AA	A	6	180	20	150	-0.8	3	3.5	0
IT	-		AA	A	3	90	20	50	1	2	3.5	0
IT	UB_EIT_3_6	Pesaro	AA	A	5	80	40	180	-0.5	9	3.2	0
IT	-		AA	A	2	160	40	100	1		3.2	0
IT	UB_EIT_3_7	Pesaro	AA	E	1	1500		200	-0.5		2.8	0
IT	UB_EIT_3_8	Pesaro-Fano	AA	A	90	85	25	50	1	3	2.5	0
IT	UB_EIT_3_9	Fano	AA	A	10	100	40	180	-0.5	10	3.3	0
IT	-		AA	A	11	60	40	90	1	3	3.3	0
IT	UB_EIT_3_10	Metaurilia	AA	A	4	80	30	45	2	4.5	1	0
IT	-		AA	A	7	200	15	20	2	4.5	1	0
IT	UB_EIT_3_11	Marotta	AA	A	2	25	10	20	1		2.5	0
IT	-		AA	A	3	150	25	200	-0.5	10	2.5	0
IT	-		AA	A	1	640		65	-1.3	2	2.5	0
IT	-		AA	A	30	250	60	230	-0.5	10	3	0
IT	UB_EIT_3_12	Senigallia	AA	A	4	80	20	150	-0.5	10	3	0
IT	-		AA	A	6	70	20	60	1.5	3	3	0
IT	-		AA	A	34	60	45	160	1.5	3	3	0
IT	-		AA	A	7	80	25	150	-0.5	10	3	0
IT	UB_EIT_3_13	Marina di Mon	AA	A	4	110	30	150	-1	10	3.7	0
IT	UB_EIT_3_14	Rocca Priora di	AA	A	11	70	25	90	-1	3	3	0

Country	Filename	Name of site	Purpose	Type	N_{umber}	Length L_s [m]	Gap G [m]	Dist. X [m]	Freeboard R_c [m]	Width B [m]	Depth h [m]	Tide tr [m]
IT	UB_EIT_3_15	Falconara Marit	AA	A	65	70	25	130	1.5	3		0
IT	UB_EIT_3_16	Ancona	AA	A	1	100		20	0.8	3		0
IT	-		AA	A	1	100		65	1.5	3		0
IT	UB_EIT_3_18	Sirolo and Num	AA	A	2	200		50	1.3	3	4	0
IT	-				2	200		50	-1	4	4	0
IT	UB_EIT_3_19	Scossicci-Porto	AA	A	20	65	30	45	1.3	3	4.5	0
IT	-		AA	A	7	150	30	100	-1	5	4.5	0
IT	UB_EIT_3_20	Porto Potenza	AA	A	4	75	40	65	1.5	4		0
IT	-		AA	A	11	85	35	110	1.5	4		0
IT	-		AA	A	11	40	25	30	1.5	4		0
IT	UB_EIT_3_21	Fontespina-Port	AA	A	14	50	35	60	1	4	3	0
IT	-		AA	A	1	600		120	-1	3	3	0
IT	-		AA	A	8	100	35	50	1	4	3	0
IT	UB_EIT_3_22	Porto S.Elpidio	AA	A	2	750		40	-1	3	2.5	0
IT	-		AA	A	6	80	40	160	-0.5	4.5	4	0
IT	UB_EIT_3_23	From Lido di Fe	AA	A	62	60	35	100	1.5	3	2.5	0
IT	UB_EIT_3_24	Marina Palmen	AA	A	8	100	40	150	-0.5	4.5	3	0
IT	UB_EIT_3_25	Marina di Altido	AA	A	7	80	25	100	1.5	4	4	0
IT	-		AA	A	3	70	30	70	1.5	4	3.5	0
IT	UB_EIT_3_26	Pedaso	AA	A	3	100	30	80	-0.5	10	4	0
IT	-		AA	A	4	55	30	80	1	3	4	0
IT	-		AA	A	2	100	10	15	1	3	4	0
IT	-		AA	A	1	25	10	15	1	3	4	0
IT	UB_EIT_3_27	Marina di Camp	AA	A	9	80	35	120	1	3	4	0
IT	-		AA	A	1	100		120	-0.5	10	4	0
IT	UB_EIT_3_28	Cupramarittima	AA	A	50	80	35	150	1.2	3	3.2	0
IT	-		AA	A	5	150		150	-0.5	10	3.2	0
IT	UB_EIT_3_29	Grottammare	AA	A	10	90	20	150	-0.9	3	2.7	0
IT	-				4	75	30	75	0.8	3	2.8	0
IT	UB_EIT_3_30	Grottammare	AA	E	1	550		50	0	4	1.6	0
IT	-		AA	E	3	250	50	150	-0.5	10	3	0
IT	UB_EIT_3_31	San Benedetto	AA	A	9	80	15	170	-0.9	3	2.5	0
IT	-		AA	A	36	100	35	120	1.2	3	2.5	0

Country	Filename	Name of site	Purpose	Type	N_{umber}	Length L_s [m]	Gap G [m]	Dist. X [m]	Freeboard R_c [m]	Width B [m]	Depth h [m]	Tide tr [m]
IT	UB_EIT_6_1	Bisceglie	AA	A	5	60	30	67	-0.15	10	3.3	0.5
IT	UB_EIT_6_2	Bisceglie	AA	A	2	185	72	90	0.8	3.7	4.2	0.5
IT	UB_EIT_6_3	Bari	AA	A	15	100	30	100	1	12	4	0.5
IT	UB_EIT_6_4	Brindisi	AA	A	6	125	20	100	2.5		3	0.5
IT	UB_WIT_11_1	Marina di Mass	AA	E	10	200	30	90			5	0.4
IT	UB_WIT_11_2	Marina di Pisa	AA	E	10	200	20	50			3	0.4
IT	UB_WIT_11_3	Prato_Ranieri	AA	A	8	60	20	20				0.4
IT	UB_WIT_11_4	Follonica	AA	A	8	72	35	40				0.4
IT	UB_WIT_12_1	Ventimiglia	AA	A	3	200	115	60				0
IT	UB_WIT_12_2	Bordighera	AA	A	7	200	20	65				0
IT	UB_WIT_12_3	San Remo	AA	E	8	115	30	90				0
IT	UB_WIT_12_4	Arma di Taggia	AA	A	7	60	80	60				0
IT	UB_WIT_12_5	Santo Stefano	AA	A	10	105	60	70				0
GR	AUTH_GR_002	St. Nikolaos	AA	A	3	80	25	63	1.2		3	0.5
GR	AUTH_GR_003	Lakopetra	AA	A	3	70	40	125	0.7	4	3.3	1.2
GR	AUTH_GR_004	Paphos, Cyprus	AA	A	1	110		70	0	5	4.5	
GR	AUTH_GR_005	Alaminos, Larn	AA	A	4	140	25	140	0.5	3.7	3	
ES	UPC_ES_001	Playa de Cubel	AA	A	3	130		150	0.7	5	2.5	0.25
ES	-				2	100		90	-0.25	5	2.25	0.25
ES	UPC_ES_002	Playa de Altaful	AA	A	1	116		180	0.5	5	4	0.25
ES	UPC_ES_003	Playa del Puert	AA	A	2	80		140	0	12.5	1.5	0.25
ES	UPC_ES_004	Playa de Vinaro	AA	A	2	125		170	0.5	17.5	4.5	0.3
ES	UPC_ES_005	Playas Comín	AA	A	3	110		140	0.5	5.5	2.3	0.3
ES	UPC_ES_006	Playa de Altea	AA	A	1	220		200	0.2	12	5	0.3
ES	UPC_ES_007	Playa de Camp	AA	E	1	98		130	0.5	5	4	0.3
ES	-				1	72		135	-1	7	3.25	0.3
ES	-				1	270		130	-3	4	3.75	0.3
ES	UPC_ES_008	Playa del Posti	AA	A	1	160		270	-2	2.5	4	0.3
ES	UPC_ES_009	Playa de Los Al	AA	A	3	177		190	-0.3	4	2.25	0.4
ES	UPC_ES_010	Playa de la Erm	AA	A	1	115		70	-1.5	7	2.5	0.4
ES	UPC_ES_011	Playa del Rihu	AA	E	3	100		145	0	18	3.75	0.4
ES	UPC_ES_012	Playa de Ponie	AA	A	1	220		125	-0.2	15	4	0.4
ES	UPC_ES_013	Playa de La Ga	AA	E	2	135		200	0.5	10	7.5	0.6

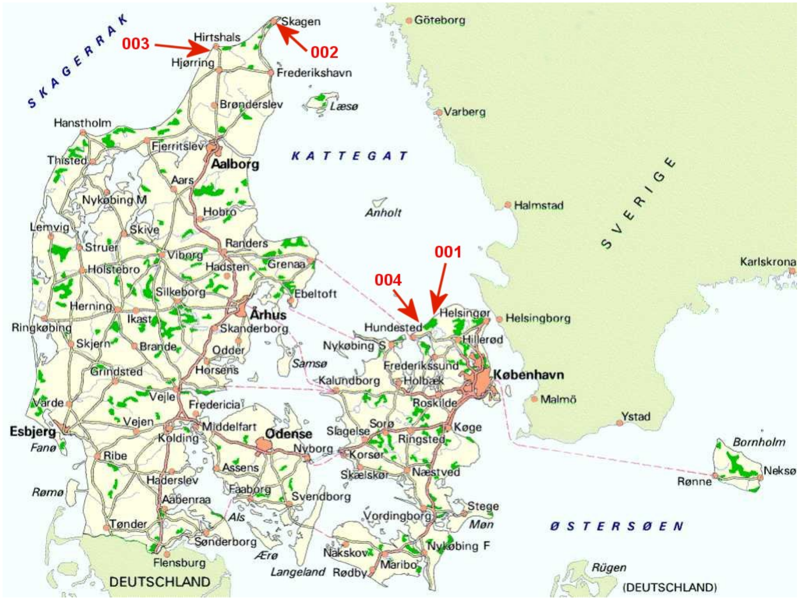
Country	Filename	Name of site	Purpose	Type	N_{umber}	Length L_s [m]	Gap G [m]	Dist. X [m]	Freeboard R_c [m]	Width B [m]	Depth h [m]	Tide tr [m]
ES	UPC_ES_014	Playa del Zapill	AA	E	1	412		170	-2.5	3	5.5	0.6
ES	-				1	212		150	0.5	5.5	5.5	0.6
ES	UPC_ES_015	Playas de Agua	AA	A	1	110		170	0.5	5.5	4.5	0.6
ES	UPC_ES_016	Playa de Castel	AA	A	3	97		30	0.5	5	6	0.7
ES	UPC_ES_017	Playa de Torren	AA	A	1				0	5		0.7
ES	UPC_ES_018	Playa de Fuent	AA	A	1	180		145	0	8	6.5	0.7
ES	UPC_ES_019	Playas del Rinc	AA	A	1	197		230	0.5	5.5	5.25	0.8
ES	UPC_ES_020	Playas del Palo	AA	A	4	135		125	-1.75	5	3.9	0.8
ES	-				3	90		125	1.25	20	3.9	0.8
ES	-				1	300		125	1.25	13	4	0.8
ES	UPC_ES_021	Playa de Bena	AA	A	1	200		160	0.5	11.5	7	0.8
ES	UPC_ES_022	Playa del Tablill	AA	E	1	135		80	0	11	4	3
ES	UPC_ES_023	Playa del Ancla	AA	E	1	90		70	0	16	6	3
ES	UPC_ES_024	Playa de La Laj	AA	A	1	200		140	1	20	6	2.7
ES	UPC_ES_025	Playa de Baja	AA	E	3	50	50	125	0.5	6	7	2.7
ES	-				3	50	50	125	-1.5	6	7	2.7
ES	UPC_ES_026	Playa de Güima	AA	E	2	175		85	2.5	10	8.5	2.5
ES	-				1	100		85	2.5	10	8.5	2.5
ES	UPC_ES_027	Playa de Fañab	AA	E	4	60		200	-1	16	5	2.5
ES	-				3	120		200	2	15	5	2.5
ES	UPC_ES_028	Playa Jardín	AA	E	1	560		100	-2.6	13	5	2.7
UK	UoS_UK_001	Elmer West Su	AA	A	3	140		130	0	4	6	6.3
UK	-				5	80		130	0	4	6	6.3
UK	UoS_UK_002	Monk's Bay det	AA	E	1	75		40	2.2	4	1.5	3.6
UK	UoS_UK_003	Milford on Sea	AA/BB	E	1	80			2.5			2
UK	UoS_UK_005	Sidmouth detac	AA	A	2				3.6		4.5	4.2
UK	UoS_UK_013	Rhos on Sea	AA	A	1	250		150				7
UK	UoS_UK_016	Leasowe Bay	AA	E	1	240		140		5		10
UK	-				1	210		140		6		10
UK	UoS_UK_025	Happisburgh to	AA	A	16	220	280	200				2
USA	UCAinventory_non_EU	Whinthrop	AA	A	6	100	30	305	1	7.5	3	1.5
USA	UCAinventory_non_EU	Colonial beach,	AA	A	4	61	46	64			1.2	1.2
USA	UCAinventory_non_EU	Colonial beach,	AA	A	3	64	26	46			1.2	1.2

Country	Filename	Name of site	Purpose	Type	N_{umber}	Length L_s [m]	Gap G [m]	Dist. X [m]	Freeboard R_c [m]	Width B [m]	Depth h [m]	Tide tr [m]
USA	UCAinventory_non_EU	Elms Beach	BB	A	3	47	53	44			0.75	1
USA	UCAinventory_non_EU	Elk Neck State	BB	A	4	15	15		0.5		0.75	1
USA	UCAinventory_non_EU	Terrapin Beach	BB	A	4	23	23				0.75	1
USA	UCAinventory_non_EU	Eastern Neck	BB	A	26	31	23		0.5	12.5	0.45	1
USA	UCAinventory_non_EU	Bay ridge	AA	A	11	31	31	43	1			1
USA	UCAinventory_non_EU	Redington Shor	AA	A	1	100		104	0.5	7.5		0.9
USA	UCAinventory_non_EU	Holly beach 1	AA	A	6	49	91	70			2.5	0.9
USA	UCAinventory_non_EU	Holly beach 2	AA	A	76	47	92	122	1	7.5	1.5	0.9
USA	UCAinventory_non_EU	Grand Isle	AA	A	4	70	21	107			2	0.6
USA	UCAinventory_non_EU	Lakeview Park	AA	A	3	76	49	143	2.4	4.2	2.4	1.8
USA	UCAinventory_non_EU	Presque Isle 1	AA	A	3	38	62	60	1		1	
USA	UCAinventory_non_EU	Presque Isle 2	AA	A	55	46	107	92	1		2	
USA	UCAinventory_non_EU	Lakeshore Park	AA	A	3	38	61	120			2.1	
USA	UCAinventory_non_EU	East Harbor	AA	A	4	46	105	170	1		2.3	
USA	UCAinventory_non_EU	Maumee Bay	AA	A	5	61	76		1.5	7.5	1.3	
USA	UCAinventory_non_EU	Sims Park	AA	A	3	38	49				2.5	
USA	UCAinventory_non_EU	Forest Park	AA	E	3	63	75	95	2.1	4	3	
USA	UCAinventory_non_EU	Klode Park	AA	E	3	24	38	49	2	4	2	
USA	UCAinventory_non_EU	Venice, Californ	AA	A	1	180		370	1.5	7.5		2.5
USA	UCAinventory_non_EU	Haleiwa Beach	AA	A	1	49		90			2.1	0.9
USA	UCAinventory_non_EU	Sand Island	AA	A	3	21	23					0.8

A.4 Country-specific geometry of LCS's

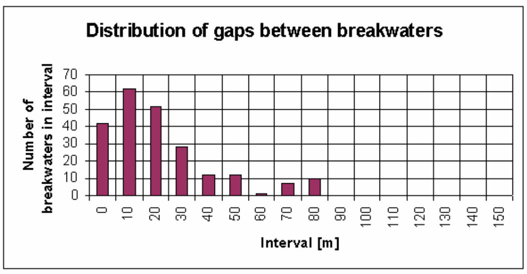
In the subsequent tables maximum, minimum and average/typical information is specified for the LCS's in each country. Parameters are specified only for the detached LCS part of the scheme.

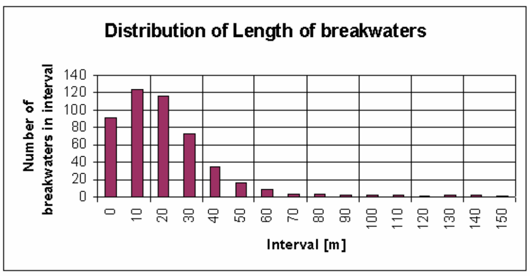
A.4.1 LCS's in Denmark



Information collected by partner	DHI
No. of investigated sites	4 sites for the brief questionnaire, simple statistics for 496 breakwaters
Purpose	Beach protection to protect land or buildings
Construction types	Mainly simple groynes and detached breakwaters used for beach protection
Length of segments [m]	5 (Minimum), 29 (Typical), 169 (Maximum)
Gap [m]	4.2 (Minimum), 26.5 (Typical), 88.7 (Maximum)
Distance to shoreline [m]	25 (Minimum), 50 (Typical), 75 (Maximum)
Freeboard [m]	0.0 (Minimum), +1.0 (Typical), +1.3 (Maximum)
Crest width [m]	2.0 (Minimum), 2.5 (Typical), 3.0 (Maximum)
Water depth [m]	1.0 (Minimum), 1.1 (Typical), 1.2 (Maximum)
Water level variations [m]	0.4 (Typical), 2.0 (Maximum)

Along the Danish coasts about 500 breakwaters exist. These breakwaters are not designed as Low Crested Structures. However in storms the breakwaters are overtopped due to the fact that the crest freeboard is about +1 to +2 meters. The construction material is mainly rubble mound. In addition to the brief questionnaire a simple statistics for 496 breakwaters were made. The gap and the length of the structures were calculated from existing UTM-coordinates (the data were provided by KDI). All gaps less than 4 m or larger than 100 m were ignored.





A.4.2 LCS's in the Netherlands

Information collected by partner	INF
No. of investigated sites	6 sites for the brief questionnaire
Purpose	Various
Construction types	Various
Length of segments [m]	-
Gap [m]	-
Distance to shoreline [m]	-
Freeboard [m]	-
Crest width [m]	-
Water depth [m]	-
Water level variations [m]	1.5 (Minimum), 1.6 (Typical), 2.0 (Maximum)

The Low Crested Structures in the Netherlands are rather atypical; only one structure is protecting a beach. The following pictures are from Punt van Reide (structure no. 001), where the structures were built for ecological reasons.

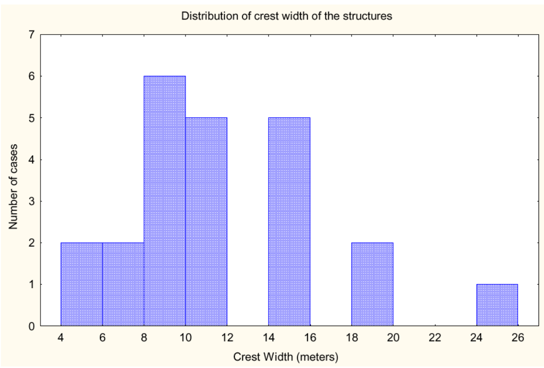
A.4.3 LCS's in Italy

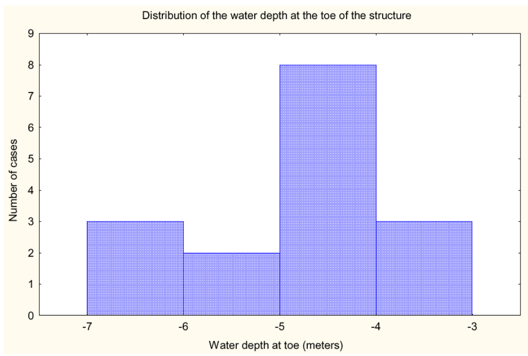
Part provided by UR3/MOD

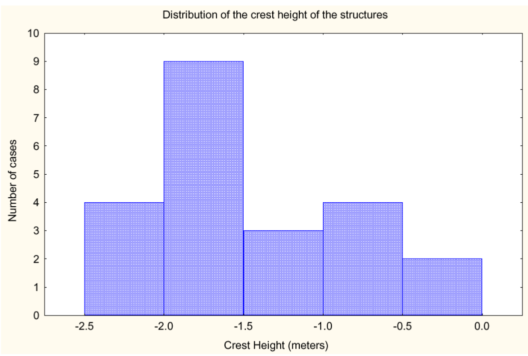
Information collected by partner	UR3/MOD
No. of investigated sites	18 sites for the brief questionnaire
Purpose	Beach protection to protect land or buildings
Construction types	Mainly detached breakwaters used for beach protection
Length of segments [m]	30 (Minimum), - (Typical), 3000 (Maximum)
Gap [m]	-
Distance to shoreline [m]	55 (Minimum), ~150 (Typical), 300 (Maximum)
Freeboard [m]	-1.75 (Minimum), -1.1 (Typical), 0.0 (Maximum)
Crest width [m]	6 (Minimum), 13 (Typical), 20 (Maximum)
Water depth [m]	2.5 (Minimum), 4.3 (Typical), 6.5 (Maximum)
Water level variations [m]	0.4 (Minimum), 0.5 (Typical), 1.0 (Maximum)

Part provided by UB

Information collected by partner	UB
No. of investigated sites	56 sites for the brief questionnaire
Purpose	Beach protection to protect land or buildings
Construction types	Mainly detached breakwaters used for beach protection
Length of segments [m]	20 (Minimum), 100 (Typical), 1500 (Maximum)
Gap [m]	~0 (Minimum), 30 (Typical), 102 (Maximum)
Distance to shoreline [m]	0 (Minimum), 100-150 (Typical), 260 (Maximum)
Freeboard [m]	-1.0 (Minimum), -1.0 or +1.5 (Typical), +2.0 (Maximum)
Crest width [m]	2 (Minimum), 4 or 10 (Typical), 12 (Maximum)
Water depth [m]	1.6 (Minimum), 3.0 (Typical), 5.0 (Maximum)
Water level variations [m]	0 (Minimum), 0.3 (Typical), 0.8 (Maximum)







A.4.4 LCS's in Greece

Information collected by partner	AUTH
No. of investigated sites	4 sites for the brief questionnaire, 2 of them on Cyprus
Purpose	Beach protection to protect land or buildings
Construction types	Detached breakwaters used for beach protection
Length of segments [m]	70 (Minimum), 100 (Typical), 140 (Maximum)
Gap [m]	20 (Minimum), 30 (Typical), 40 (Maximum)
Distance to shoreline [m]	63 (Minimum), 100 (Typical), 140 (Maximum)
Freeboard [m]	0.0 (Minimum), +0.6 (Typical), +1.2 (Maximum)
Crest width [m]	3.0 (Minimum), 5 (Typical), 7.5 (Maximum)
Water depth [m]	2 (Minimum), 3.5 (Typical), 4.5 (Maximum)
Water level variations [m]	-

2 detailed questionnaires have been completed. No. 003 (for location see map above) Lakopetra, Ahaia, and no. 004 Paphos on Cyprus.

No. 003 consists of tree successive detached breakwaters parallel to the shoreline. The scheme was constructed in 1992 in order to protect and stabilize a 300 m sandy beach section that lies in front of a hotel. The beach was particularly vulnerable to wave attack which posed hazards to the swimmers.

No 004 is a detached breakwater parallel to the shoreline. The beach that extends in front of a hotel is rocky hence it was inaccessible to swimmers. To overcome this deficiency a sufficient amount of rocky mass was excavated and removed and an artificial beach pocket was created applying sand nourishment. Considering that the coastline in the area is subject to significant wave action, the breakwater was constructed in order to maintain and enhance the pocket.

A.4.5 LCS's in Spain

Information collected by partner	UPC
No. of investigated sites	28 sites. 21 at the Spanish Mediterranean Coast, 7 at the Canary Islands
Purpose	Beach protection to protect land or buildings
Construction types	Mainly detached breakwaters used for beach protection
Length of segments [m]	25 (Minimum), 140 (Typical), 412 (Maximum)
Gap [m]	-
Distance to shoreline [m]	25 (Minimum), 130 (Typical), 270 (Maximum)
Freeboard [m]	-3.6 (Minimum), +0.5 (Typical), +2.5 (Maximum)
Crest width [m]	4 (Minimum), 5 or 15 (Typical), 20 (Maximum)
Water depth [m]	1 (Minimum), 5 (Typical), 9 (Maximum)
Water level variations [m]	0.25 (Minimum), 0.5 or 2.7 (Typical), 3.0 (Maximum)

The structures at the Spanish Mediterranean Coast and the structures at the Canary Islands are quite different, e.g.

A) The tidal range on the Canary Islands is large (tidal ranges of 2.5 to 3m). On the Spanish Mediterranean Coast it is below 1m.
B) The structures on the Canary Islands are built in large water depth (5 to 9m). On Spanish Mediterranean Coast the water depth is typical below 5m.
C) The slope of the beach is typical more steep on the Canary Islands.

Playa de Cubelles (Beach of Cubelles), Barcelona, is shown on the picture below.

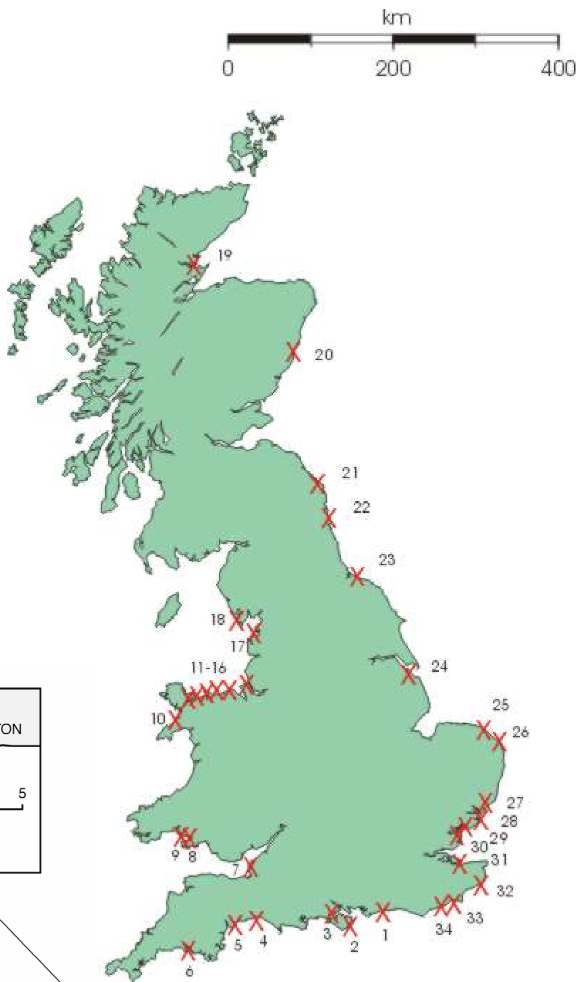
Playa de Fañabé (Fañabé Beach), Tenerife, Canary Islands, is shown on the picture below.

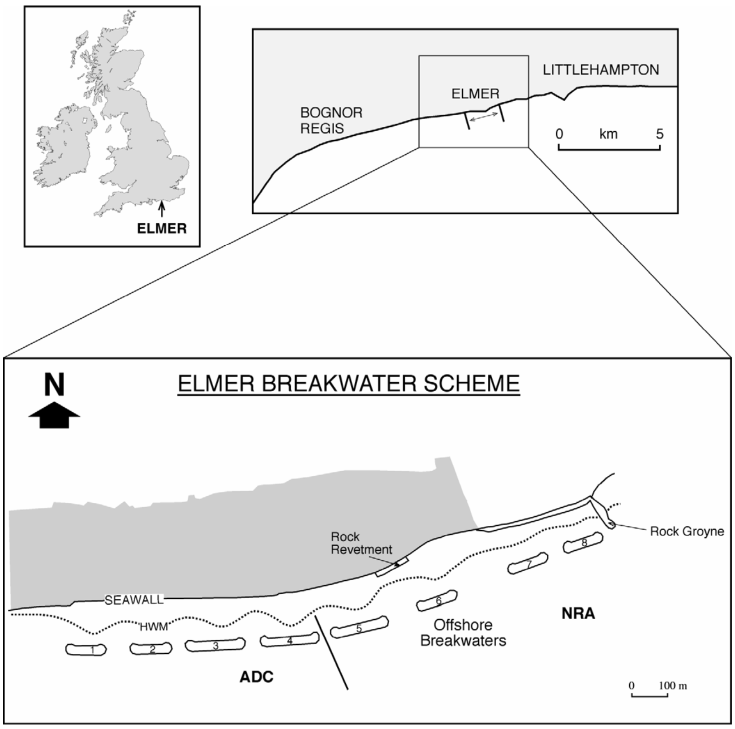
A.4.6 LCS's in UK

Information collected by partner	UoS
No. of investigated sites	34 sites for the brief questionnaire
Purpose	Beach protection to protect land or buildings
Construction types	Mainly simple groynes and detached breakwaters used for beach protection
Length of segments [m]	75 (Minimum), 500 (Maximum)
Gap [m]	-
Distance to shoreline [m]	0 (Minimum), 200 (Maximum)
Freeboard [m]	0.0 (Minimum), +7.3 (Maximum)
Crest width [m]	4.0 (Minimum), 6.0 (Maximum)
Water depth [m]	0.0 (Minimum), 6.0 (Maximum)
Water level variations [m]	2.0 (Minimum), 11.0 (Maximum)

In UK a broad range of structures have been investigated and included in the inventory. This includes detached breakwaters, a lot of schemes with fishtail groynes, L-shaped groynes, and a scheme with cliff stabilization structures. Therefore the detail of the structural geometry in the brief questionnaire is generally limited except for the Elmer scheme.

The Elmer scheme consists of 8 shore parallel structures and a terminal rock armour groyne. Each structure is approximately 6m high and 4m wide. They are situated at an average of 130m from the sea wall. There is a great deal of detailed information about the scheme, the structures, the sedimentology, the hydrodynamics, and the beach morphology.





Appendix B Prototype observations in Denmark

This appendix describes the Danish DELOS study site locations with respect to geometrical layout, materials, hydrodynamic conditions and ecological investigations. The description of geometrical layout and materials are based on existing measurements, statistics and literature. The hydrodynamic conditions are based on existing knowledge and new calculations of wave climate by the structures. At the study locations new ecological field investigations have been carried out. The text has earlier partly been published through the DELOS project as part of the Work Package 2.5, Delivery 58, see Kramer and Dinesen (2004). For further information regarding prototype experience regarding LCS's in EU, see Lamberti *et al.* (2005).

B.1 Introduction

On the basis of existing knowledge about the Danish breakwaters and the information collected within DELOS WP1.1 "Inventory of existing LCS" three locations in Denmark was selected for investigations. The study sites on the West coast of Denmark were Hirtshals Harbour and Lønstrup. The third study site is on the East coast near the spit of Denmark close to Skagen.

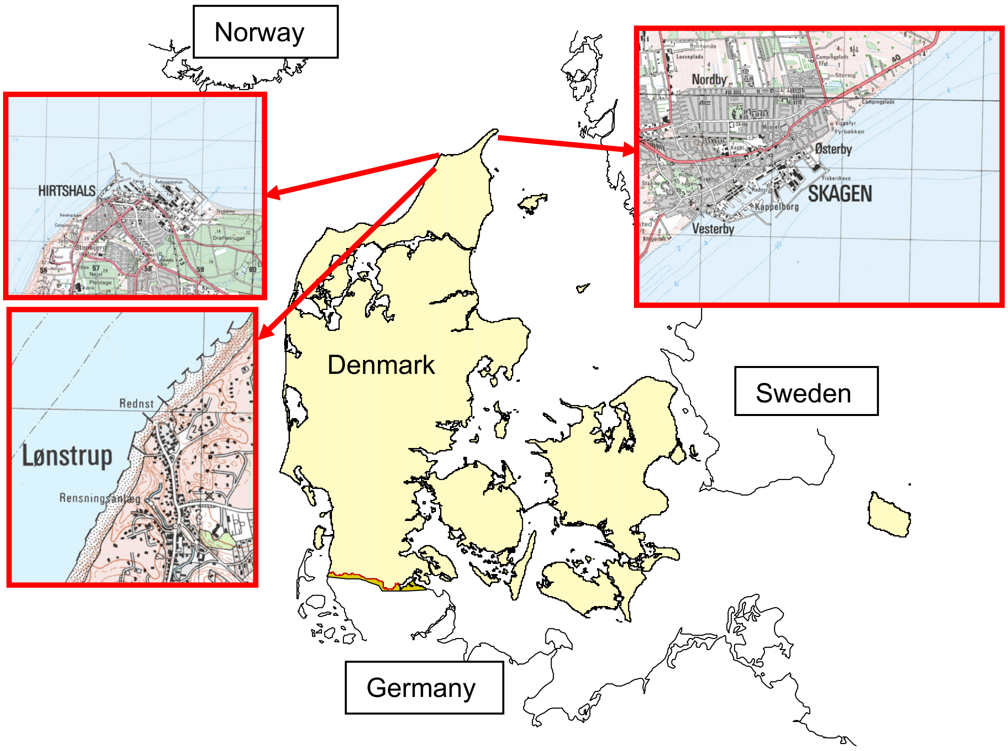


Figure B.1. Position of study locations for ecological investigations.

B.1.1 Skagen

Skagen harbour is located just south of the Skaw spit ('Grenen' seen on the map below) facing Kattegat. The map covers an area of app. 10.5*8.3 km^2. Information on the breakwaters was provided by KDI (The Danish Coastal Authority, 1995).

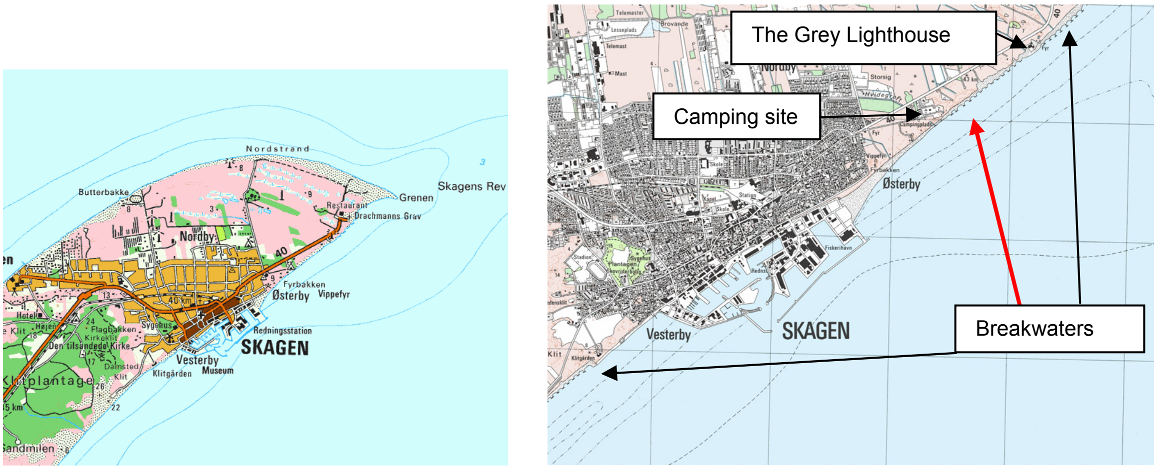


Figure B.2. Study site location at Skagen. The breakwaters for the ecological investigations are located between the Camping site and the Grey Lighthouse, see the red arrow in the right figure.

Figure B.3 Pictures of the study site at Skagen (Left picture from Kystinspektoratet, 2001).

The breakwaters are built to protect the coast from the waves generated in Kattegat (the water body just north of Sealand, bounded to the east by Sweden and to the west by Jutland). Many different constructions have been made, rebuilt and even removed such that several construction schemes have been implemented over the years. Regular beach nourishment is performed on the beach landward of the breakwaters. The nourished sand is eroded continuously such that the sand supplied north of the harbour (except for that very close to the harbour) ends up around the Skaw spit.

The area is a very popular tourist spot in Denmark and the area relies heavily – as it is – on tourism. Furthermore the harbour is very important with respect to landing, processing and distribution of pelagic fish (see www.ifm.dk).

B.1.2 Lønstrup

The map below shows the location of Lønstrup. Both north and south of this site a system of breakwaters is located, see aerial photo below. Information on the breakwaters is obtained from Laustrup and Madsen (1994).

The breakwaters were built to protect the small village Lønstrup located near the sea and the adjacent beaches from the ongoing coastal erosion caused by the North Sea waves. The coast at Lønstrup will if unprotected erode 1.5 m/year. In fact during a storm back in 1981 some places were eroded up to 15 m. This event initiated the construction of a protection scheme in 1982/1983. The area of interest is being nourished with 20.000-30.000m^3/y sand between the coast and the breakwaters. The area is a popular tourist and nature spot in Denmark and many summerhouses are located here.

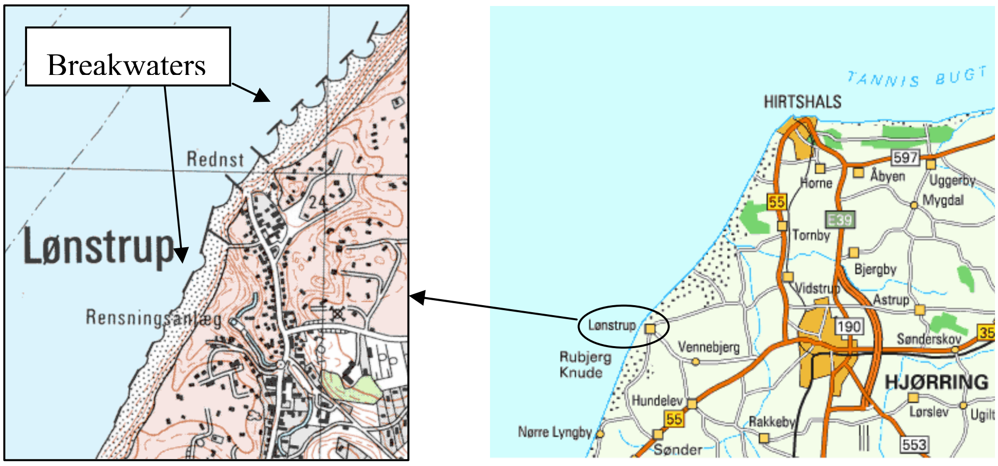


Figure B.4. Study site location at Lønstrup.

Figure B.5. Picture of the study site at Lønstrup during stormy weather. Photo by Poul Jacobsen.

B.1.3 Hirtshals Harbour

Hirtshals Harbour is one of the largest fishing and ferry ports in Denmark. It is built at one of the very exposed points of the northwest coast of Denmark. The defence structures are built to protect the harbour basins from large waves (and not for beach protection).

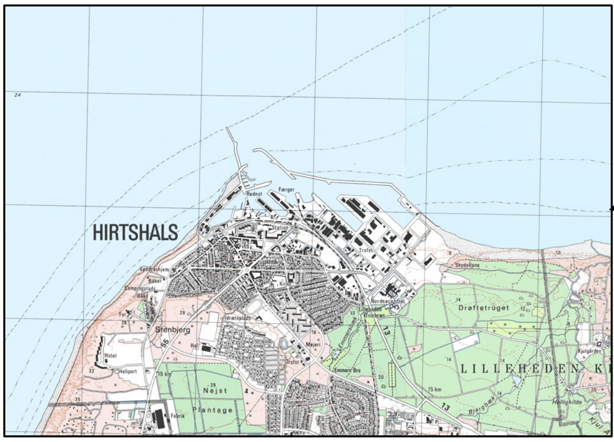
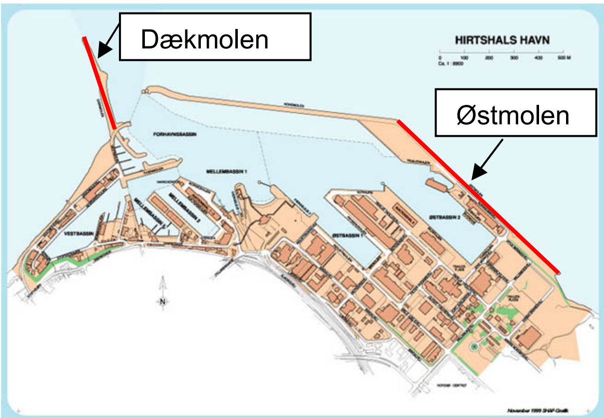


Figure B.6. Study site location at Hirtshals.

The ecological investigations concentrated about the Eastern breakwater "Østmolen" and Western breakwater "Dækmolen".



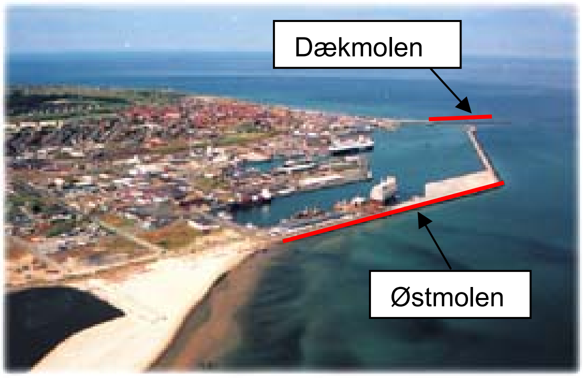


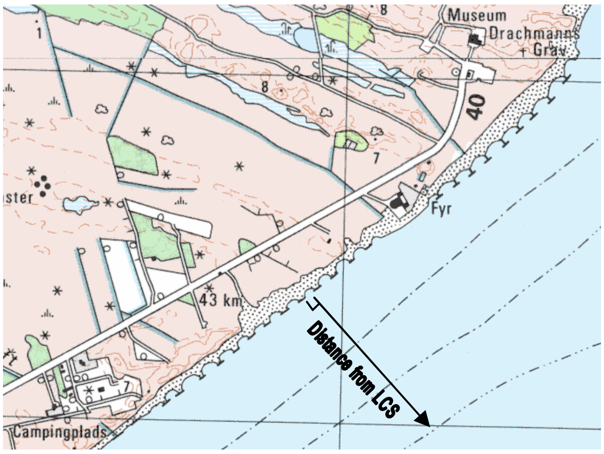
Figure B.7. Location of the breakwaters "Østmolen" and "Dækmolen".

Figure B.7 is from the Internet "www.hirtshalshavn.dk/" where more information about the harbour can be found (in Danish).

B.2 Skagen

Between the Grey Lighthouse and the spit, eight breakwaters have been constructed with gaps between them ranging from 20 to 30m (60m between number five and six, though). They are 35 to 45m long and 10m wide. On the stretch from the Grey Lighthouse to Skagen harbour 11 breakwaters were constructed, followed by seven T-shaped breakwaters. The gaps between the breakwaters are 25m (except between number three and four and five and six with spacing of app. 80m). The breakwaters are 42m long and 10m wide. On the southern side of Skagen harbour, south of Klitgården - seven T-shaped breakwaters have been constructed. From here the T-shaped structures are followed by five breakwaters. The distance between these individual breakwaters (the gap) is 25-30m. On average they are app. 30-35m long and 5-7m wide.

They all have circular round head and are aligned parallel to the coast. The system of breakwaters found along the coastline is located sufficiently close to the beach so that tombolo formations are generated. They are without exception connected to the coast, which properly is due to the nourishment. The water depth on the offshore facing side is approximately 1m. The steepness of the bottom slope is approx. 1:100 close to the breakwaters (up to approx. 500m from the beach).



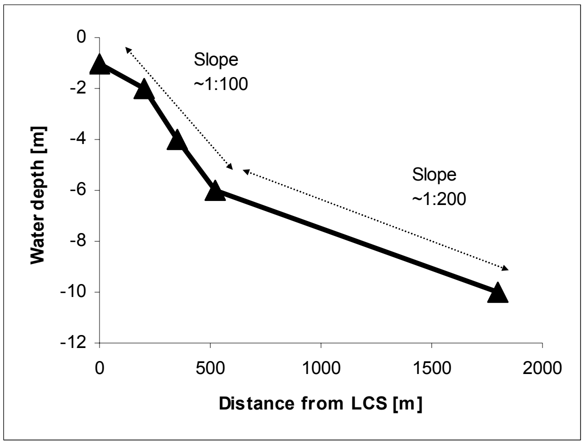


Figure B.8. Bottom topography Skagen (left: one square on the map = 1km^2).

They are all rouble mound breakwaters. A sketch of typical cross-section is shown in the figure below (from Kystdirektoratet). The level of the crest is seen to be 1 m above DNN (Dansk Normal Nul).

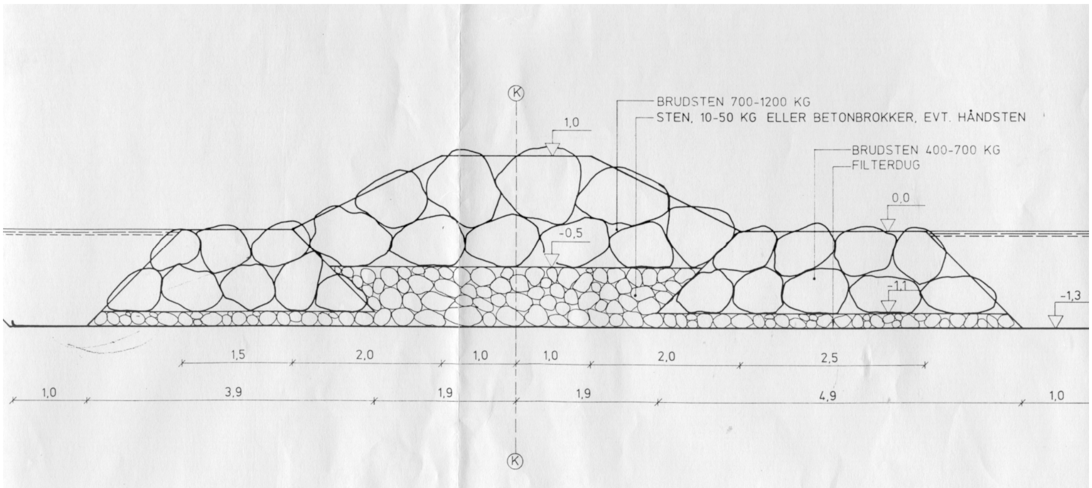


Figure B.9. Typical cross-section of Skagen breakwaters.

B.2.1 Water level variations

The difference between the water level at mean low and mean high tide is 0.3 m. Storm surge can be around 1.4m above MSL in case of storms from West. In case of storms from the East the water depth can be 0.9m below MSL (Ferskvandsdirektoratet, 1977). Typically the wind is from the West. The wind is very infrequent from the East.

B.2.2 Wind and waves

The waves will be depth limited due to the small water depth and the relative flat bottom slope in front of the breakwaters. When the waves propagate into shallow water the waves will break. The highest wave is estimated as $0.8 \cdot h = 1.9m$, where "h" is water depth including storm surge. In the following a calculation of the normal wave climate is performed.

Introduction

A wave height calculation based on fetch limitations, wind speeds and wind directions is carried out at deep water. The transformation in wave height distribution from deep water to shallow water is calculated by the point model proposed by Battjes and Gronendijk (2000). Battjes and Gronendijks model is developed for wave height distributions on shallow foreshores, and it takes account for water depth and foreshore slope. The given significant wave heights are incident significant wave heights. The wave heights does therefore not include influences of reflections from structures and beach.

Strong on-shore winds in connection with low-pressures create high water levels by the coast and thereby higher waves. The rise in water level, called *storm surge*, is more than +0.8m occurring approximately 4 times per year. During the storms the maximum wave height is increased with approximately 0.5m by the structures. It is estimated that there will be set-up due to wind and low pressure in approximately 1% of the time. Very strong on-shore storms can create very large storm-surge (up to +1.4m above MSL, and off-shore winds can give water levels -0.9m below MSL). These events hardly ever occur. The rare events are not included in the present calculation, as the focus of the following only is on the normal wave climate.

The tides at Skagen have some influence on the water depth and thereby at the waves. The mean low water level to mean high water level at springs are approximately 0.3m. However, in general the change in water level is small compared to the total water depth, and the waves will therefore only be slightly smaller during low tide and slightly higher during high tide. The following calculation of waves does not take the influence of tide into account. If the tide was taken into account a slight increase in significant wave heights can be expected.

Wind speeds and directions

Distributions of wind speed and wind directions for each month in the year can be found in Frydendahl (1971). Data are given for a number of recorded stations (usually data have been observed from light houses). In Petersen *et al.* (1980) a method based on a standard set of wind frequencies for the whole Denmark is given. A set of correction factors depending on height and surface roughness (type of landscape) is used to calculate the wind speeds at a given location.

At Skagen the wind speed distribution from Hirtshals is used to ensure consistency with the calculation for Lønstrup. The directional distribution according to Frydendahl (1991) for Hirtshals is compared to a calculated distribution by the method in Petersen *et al.* (1980). In Figure B.10 (left) it is seen that the two distributions show the same distribution.

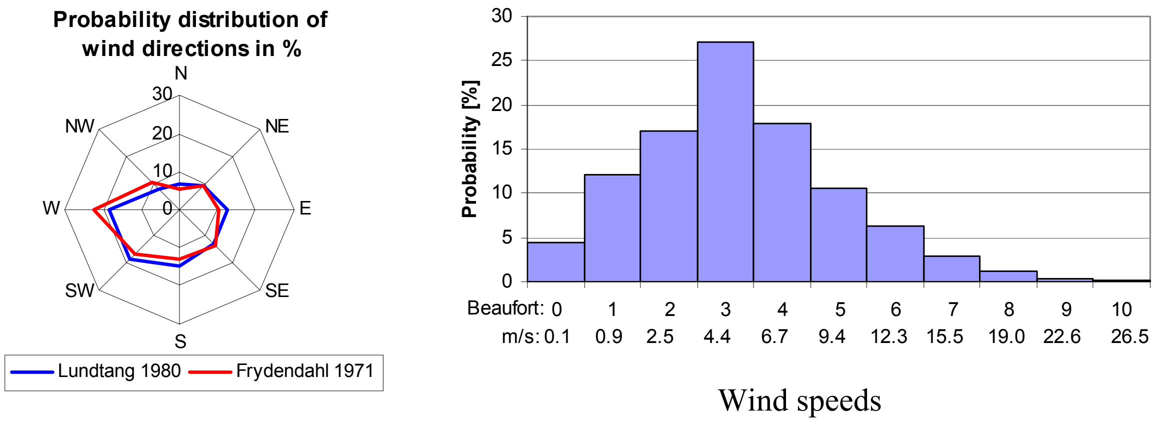


Figure B.10. Distribution of wind at Skagen. Directions (left), and speed (right, according to Frydendahl 1971).

Fetch limited wave heights at *deep water*

Lønstrup is located on the West coast and Skagen on the East coast of Denmark, see Figure B.11. At Lønstrup waves coming from the West can travel and grow over the long distance from England. For Skagen the waves can only travel and grow over the short distance to Sweden. The most frequent (and strong) winds are also coming from Western directions. Under normal wave conditions the waves at Lønstrup will therefore be larger than at Skagen.

Figure B.11. Illustration of fetch limitations .

For Skagen the fetch for waves coming from E is 75km. For SE and NE the fetch is approximately 150km. Waves coming from S will refract and hit the structures and are therefore also be included. To the South of Skagen are small islands and the mainland is also causing shelter from the waves. The fetch for South is therefore chosen as 75km. The method given in SPM, 1984 allows for calculation of fetch limited waves from given wind speeds. The results are given in Table B.1.

Table B.1. Wave heights and probability at Skagen for winds coming in different directions.

Wind speeds		NE (Fetch 150km)		E (Fetch 75km)		SE (Fetch 150km)		S (Fetch 75km)	
Beaufort	m/s	Prob. [%]	H_{m0} [m]	Prob. [%]	H_{m0} [m]	Prob. [%]	H_{m0} [m]	Prob. [%]	H_{m0} [m]
0	0.1	0.31	0.00	0.31	0.00	0.31	0.00	0.31	0.00
1	0.9	1.20	0.01	1.40	0.01	2.50	0.01	2.40	0.01
2	2.45	2.40	0.08	3.20	0.08	5.00	0.08	4.70	0.08
3	4.4	2.00	0.34	2.20	0.34	2.80	0.34	2.80	0.34
4	6.7	1.30	0.95	1.50	0.87	1.50	0.95	1.60	0.87
5	9.35	0.90	1.84	0.90	1.30	0.70	1.84	0.70	1.30
6	12.3	0.60	2.58	0.50	1.83	0.30	2.58	0.30	1.83
7	15.5	0.30	3.43	0.20	2.43	0.20	3.43	0.10	2.43
8	18.95	0.10	4.40	0.10	3.11	0.00	4.40	0.00	3.11
9	22.6	0.00	5.46	0.00	3.86	0.00	5.46	0.00	3.86
10	26.45	0.00	6.62	0.00	4.68	0.00	6.62	0.00	4.68

<u>Wave heights at the breakwater locations</u>

The transformation in wave height distribution from deep water to shallow water is calculated by the point model proposed by Battjes and Gronendijk, 2000. Significant wave heights cannot be larger than 0.6 multiplied by the water depth due to depth limitations. Large significant wave heights have therefore been replaced by 0.6 multiplied by the water depth.

The Battjes and Gronendijk model have been applied on the results in Table B.1 and the results have been grouped in ranges of wave heights 0-0.1m, 0.1-0.55m, and depth limited significant wave heights, see Table B.2. For winds coming over the main land (indicated as *other directions* in Table B.2 and Figure B.16) the wave heights close to the coast are assumed to be small and are therefore put in the range 0-0.1m.

Table B.2. Significant wave height distribution at the breakwater locations.

Significant wave height H_s [m]	Skagen Prob. [%]
0-0.1m, other dir.	54.4
0.0-0.1m	24.1
0.1-0.55m	9.8
0.6m (depth limited)	11.8

The wave heights at Skagen are compared to the wave heights at Lønstrup subsequent in Figure B.16.

B.2.3 Currents and sediments

Wind between SSW and W gives currents in NE direction. Wind between N and SSE gives currents in SW direction. However the current is generally from SW to NE. This information is obtained from Skagen Harbour but the information can be used as indication.

On Figure B.12 the black dots indicate the measured D_{50}. It is seen that D_{n50} is approx. 0.25mm at Skagen. It is also seen that the variation in the average grain size is very small; the average sediment size is approximately 0.25mm.

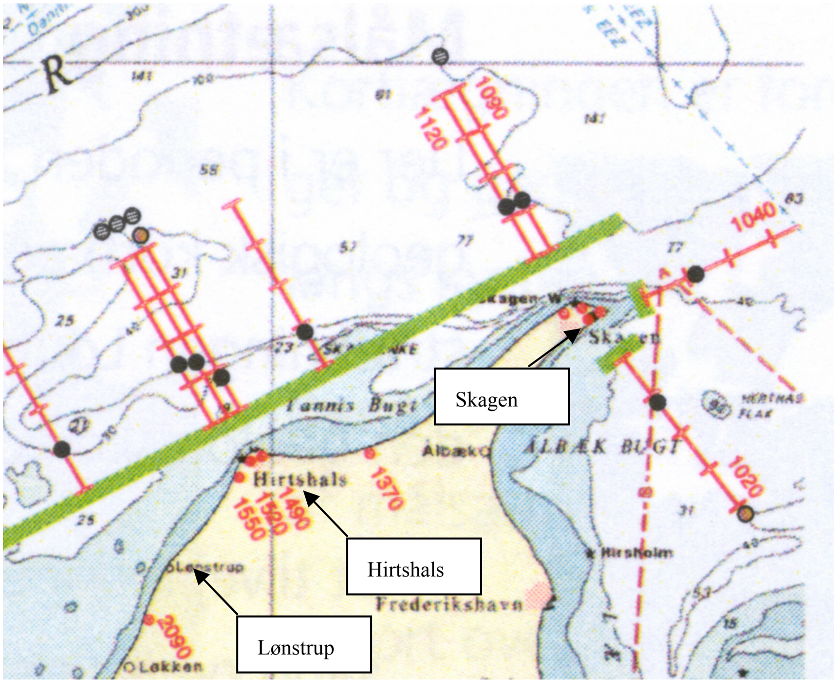


Figure B.12. Sediment size, mean diameter D_{n50}. Green line is zero, each red line-indicator is 0.2mm. Kystinspektoratet (2000).

B.2.4 Ecology

The ecological study comprised an intertidal survey horizontally along breakwaters and harbour piers and a subtidal survey vertically of the breakwater of harbour pier. Skagen Eastern Beach is among the most popular beaches in the area for recreational purposes, and the biological data obtained may have been influenced by tourists collecting e.g. larger algae on the boulders. The intertidal survey was done mainly by snorkelling and the subtidal by snorkelling and scuba-diving. The abundance and growth thickness of intertidal biota was counted on the landward and seaward sides of selected structures at depth of 0 m by semi-quantitative (MBA abundance scale, 5 replicates) and quantitative (25 cm^2 divided by 100 sub-squares, 5 replicates) methods. The quantitative data were obtained at breakwater No. 1-7, 17, and 19, see Figure B.13). The abundance of subtidal biota on frontal and shaded faces of the structure sub-units was studied on the seaward side, along 5 transects perpendicular to the shoreline at 5 depths (0 m, 0.5 m, 1 m, 1.5 m, and 2 m) at the single larger breakwater that defends the Grey Light House (No. 19). The samples are semi-quantitative counts of relative abundance at each depth along each transect (using the MBS abundance scale). All vertical transects were positioned to allow sampling down to 2 m of depth (the maximum depth of the seaward side of the structure equals or exceeds depths of 2 m). At the Grey Light House, maximum depths are about 4 m.

On the landward side of the structures, the epibiota is absent, or rarely present and then sparse. This is most likely due to the fact, that the biota are burrowed in sand and prevented access to sea water by the sand feeding activity from the landward side. The majority of marine species will not survive these events. Breakwater no. 1-7 are positioned on the shore and are rarely exposed by the sea. Here biota is rare, and includes only a few clusters of green algae (Enteromorpha spp.) and juvenile blue mussels (Mytilus edulis) that have settled recently. Breakwaters no. 17 and 19 reach into the sea to about 2 m of depth, resulting in a rather diverse flora and fauna on the seaward side of the structures. However when compared, the diversity prove larger on the subtidal part of the structures (constantly submerged parts) than on the intertidal part (at 0 m depths, irregularly submerged parts).

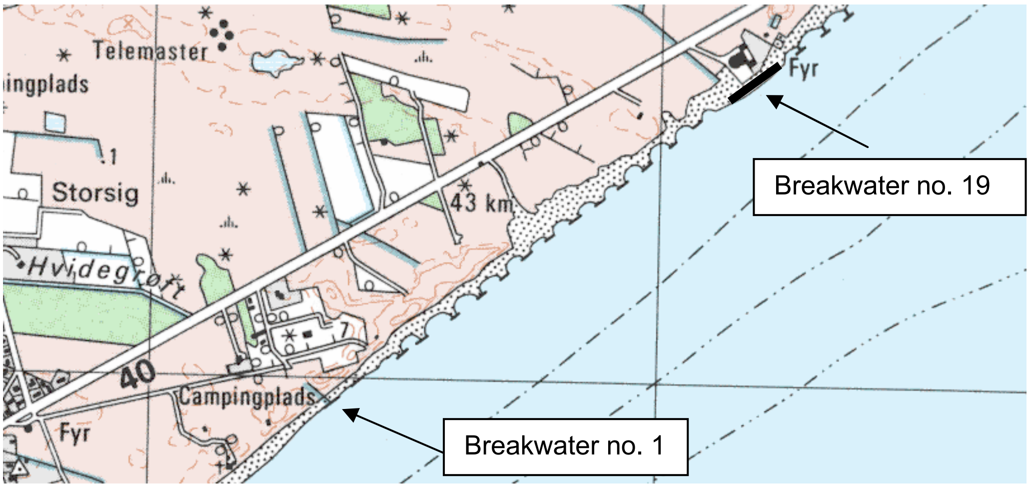


Figure B.13. The breakwaters at Skagen Eastern Beach numbered from South West to North East.

B.3 Lønstrup

The system of breakwaters at Lønstrup is 1100 m long. Each breakwater is 45 m long and separated by 45 m as well. All breakwaters have circular round heads and are aligned parallel to the coast. The system of breakwaters found along the coastline is located sufficiently close to the beach so that tombolo formations are generated. The water depth on the offshore side of the breakwaters is approximately 1 m and the steepness of the bottom slope is approx. 1:150.

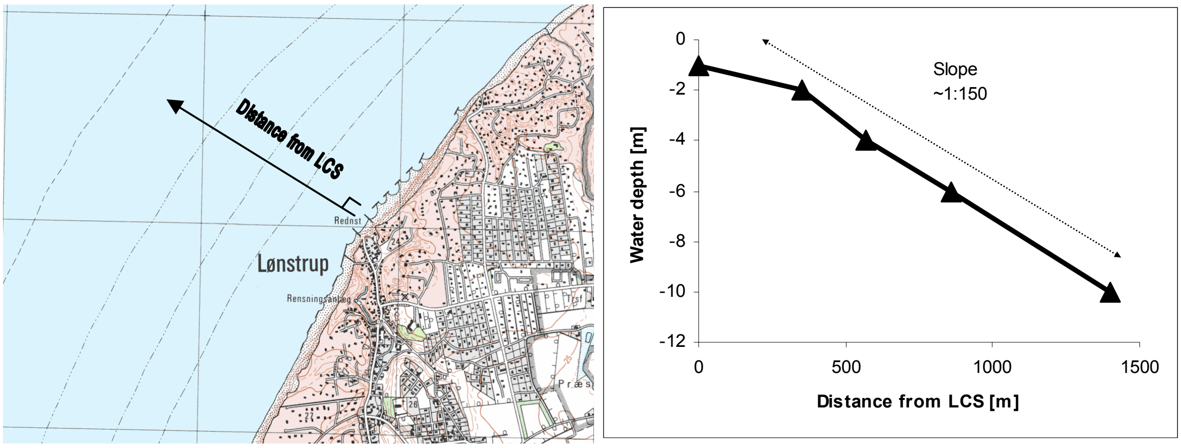


Figure B.14. Bottom topography Lønstrup (left: one square on the map = 1km^2).

All breakwaters are rouble mound breakwaters. A sketch of typical cross-section is shown in Figure 1.4. The level of the crest is seen to be 1.3 m above MWL.

B.3.1 Water level variations

The difference between the water level at mean low and mean high tide is 0.3 m. Storm surge can be around 1.5m above MSL in case of storms from West. In case of storms from the east the water depth can be 1.0m below MSL (Ferskvandsdirektoratet, 1977). Typically the wind is from the West. The wind is very infrequent from the East.

B.3.2 Wind and waves

In the winter 1981-82 waves were measured at a location 1.5km's WNW of Lønstrup at 14m's of water depth, see Kystinspektoratet (2000).

Table B.3. Statistics for Wave gauge 1002 at Lønstrup.

Occurrence	Matching wave height
1 hour/10 years	5.2 m
1 hour/year	4.4 m
10 hours/year	3.5 m
100 hours/year	2.6 m

More detailed measurements exist near Hirtshals. These measurements are presented subsequent in Chapter B.4, "Hirtshals". Those data can also be used to estimate waves at Lønstrup. Anyway close to the breakwaters the waves will be different due to reflection, refraction, diffraction and wave breaking. As at Skagen the highest wave is estimated as $0.8 \cdot h = 2.0$m, where "h" is water depth including storm surge. As preliminary estimation the typical significant wave height is roughly $0.6 \cdot h = 0.6$m.

To calculate the normal wave climate by the breakwaters the same method as used for Skagen is applied.

<u>Wind speeds and directions</u>

At Hirtshals Fyr winds have been observed in the period 1931-1960 (missing Oct. 1943 - May 1945) at the height 27m above sea level. Lønstrup is located very close to Hirtshals, and the Hirtshals data can therefore be used for Lønstrup. In Figure B.15 (left) it is seen that the directional distribution according to Lundtang 1980 and Frydendahl 1971 are the same. It is also clear that the winds are mainly coming from the West (W) and South West (SW).

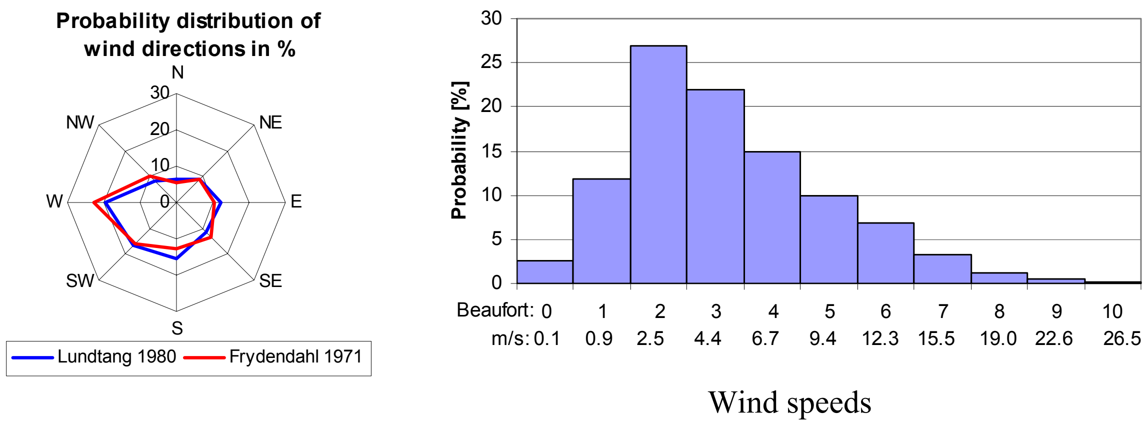


Figure B.15. Distribution of wind at Lønstrup. Directions (left), and speed (right, according to Frydendahl 1971).

<u>Fetch limited wave heights at *deep water*</u>

Winds coming from N, NW, W, and SW will produce waves at Lønstrup, which will refract and hit the coast perpendicular. It is assumed that winds from the other directions only will produce very small waves. The fetch for waves coming from N and NW is approximately 150km, and from W and SW the fetch is approximately 700km, see Figure B.11.

The wind distributions given in Figure B.15 are used together with the fetch, and the wave heights in deep water offshore Lønstrup is calculated for fetch limited waves. In Table B.4 it is seen that the calculated wave heights for wind speeds up to 4 on the Beaufort scale does not depend on the fetch. This is because the sea is fully arisen, meaning that the waves have reached an equilibrium state in which energy input from the wind is exactly balanced by energy loss.

Table B.4. Wave heights and probability at Lønstrup for winds coming in different directions.

Wind speeds		N (Fetch 150km)		NW (Fetch 150km)		W (Fetch 700km)		SW (Fetch 700km)	
Beaufort	m/s	Prob. [%]	H_{m0} [m]	Prob. [%]	H_{m0} [m]	Prob. [%]	H_{m0} [m]	Prob. [%]	H_{m0} [m]
0	0.1	0.31	0.00	0.31	0.00	0.31	0.00	0.31	0.00
1	0.9	0.90	0.01	0.90	0.01	1.20	0.01	1.50	0.01
2	2.45	1.40	0.08	1.60	0.08	3.80	0.08	4.80	0.08
3	4.4	1.10	0.34	1.70	0.34	4.80	0.34	4.50	0.34
4	6.7	0.70	0.95	1.40	0.95	4.20	0.95	2.70	0.95
5	9.35	0.50	1.84	1.40	1.84	3.40	2.15	1.50	2.15
6	12.3	0.30	2.58	1.30	2.58	2.60	4.22	0.70	4.22
7	15.5	0.20	3.43	0.80	3.43	1.30	7.42	0.30	7.42
8	18.95	0.10	4.40	0.40	4.40	0.50	9.50	0.10	9.50
9	22.6	0.00	5.46	0.20	5.46	0.20	11.79	0.00	11.79
10	26.45	0.01	6.62	0.01	6.62	0.02	14.31	0.02	14.31

<u>Wave heights at the breakwater locations</u>

Calculations follow the same procedure as for Skagen. Results are given in Table B.5 and Figure B.16. From the figure it is clear that the wave heights at Lønstrup generally are larger than at Skagen. The values in Table B.2 and Table B.5 can be used as an estimate of the differences in the normal wave climate at the two sites.

Table B.5. Significant wave height distribution at the breakwater locations.

Significant wave height Hs [m]	Lønstrup Prob. [%]
0-0.1m, other dir.	45.7
0.0-0.1m	17.4
0.1-0.55m	12.1
0.6m (depth limited)	24.9

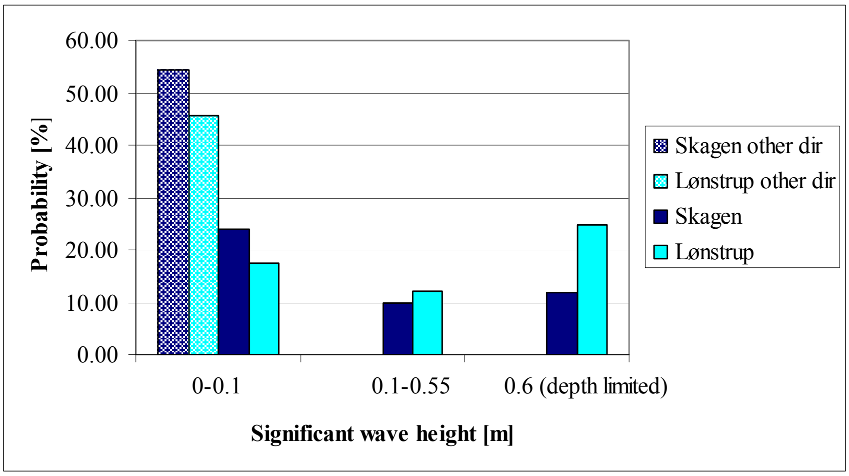


Figure B.16. Comparison of wave climate at Lønstrup and Skagen.

B.3.3 Currents and sediments

On the West coast of Northern Jutland the current and sediment transport is generally in Northern direction (indicated by arrows on Figure B.17, left). The coast is eroded and lots of material is transported from the southwest due to littoral drift on both sides of the spit. More than 1 million m^3 is deposited on the coast in the northern area of the spit every year. The sediment estimations are made from measured profile developments for water depths less than 10m.

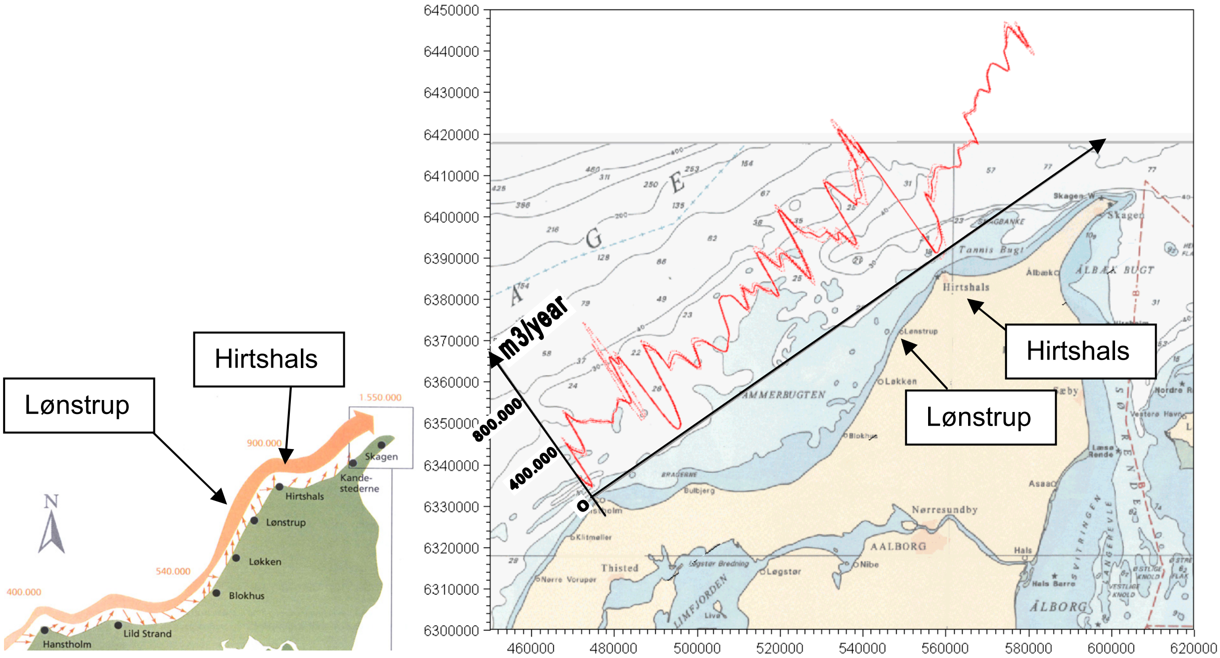


Figure B.17. Sediment transport at the West Coast in the Northern part of Denmark. Kystinspektoratet (2001).

At Lønstrup the sediment transport within 10m's water depth is approx. 600.000 m^3/year. At Hirtshals most of the sediment is deposited on the SW side of the Harbour (upstream, see arrows on Figure B.17, left) and eroded on the downstream/Eastern side. This phenomenon is also seen on the transport-curve; a big drop in the curve on Figure B.17 (figure to the right) at Hirtshals Harbour.

In Figure B.12 it is seen that D_{n50} is approx. 0.2mm at Lønstrup.

B.3.4 Ecology

The ecological study comprised an intertidal survey as described in Chapter B.2.4. The quantitative data of biota was obtained at breakwater no. 4, 5, 8, and 9, see Figure B.18. During the periods between the regular sand feeding from the landward side, tombolos are formed between the structures where algal and faunal debris are trapped and decompose, coursing anoxic conditions of the sediment below. On the landward side of structures, the biota is absent, or rarely present and then rather sparse. This is most likely due to the limited water supply on the shoreward side of the breakwaters. The structure no. 9 is positioned on the shore and rarely exposed to waves. Here biota is rare, similar to that on structure no. 1-7 at Skagen Eastern Beach (see Chapter B.2.4). Breakwater no. 4, 5, and 8 protrude into the sea and on the seaward side of the structures the intertidal biota is rather diverse and comprises both flora and fauna.

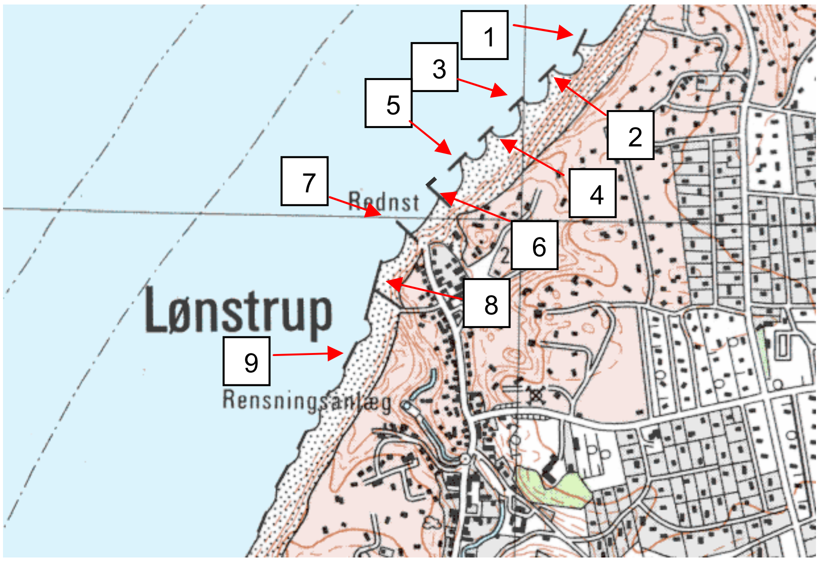


Figure B.18. The breakwaters at Lønstrup Beach numbered from North to South.

B.4 Hirtshals

As indicated in the introduction chapter (on Figure B.6, page 109) the water depth outside Østmolen is very shallow. The deepest water depth is approx. 1m in the northern end.

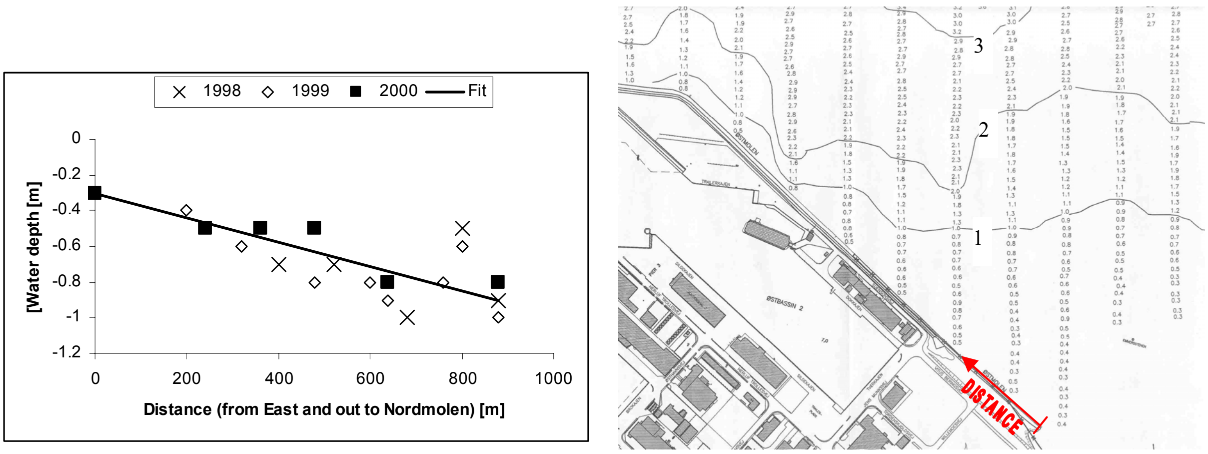


Figure B.19. Bottom topography Hirtshals Østmole. Right figure is from 2000.

All breakwaters are rouble mound breakwaters. A sketch of typical cross-section is shown in Figure B.20. The level of the crest is seen to be 3.9m above DNN (which is not a LCS). The armour layer is made of one to eight ton quarry rock. From the weight the diameter of the material can be calculated to 0.7-1.4m. The grading of the material is not known. However, the nominal diameter is estimated as the average of the smallest and the largest diameter, which gives D_{n50}=1.1m.

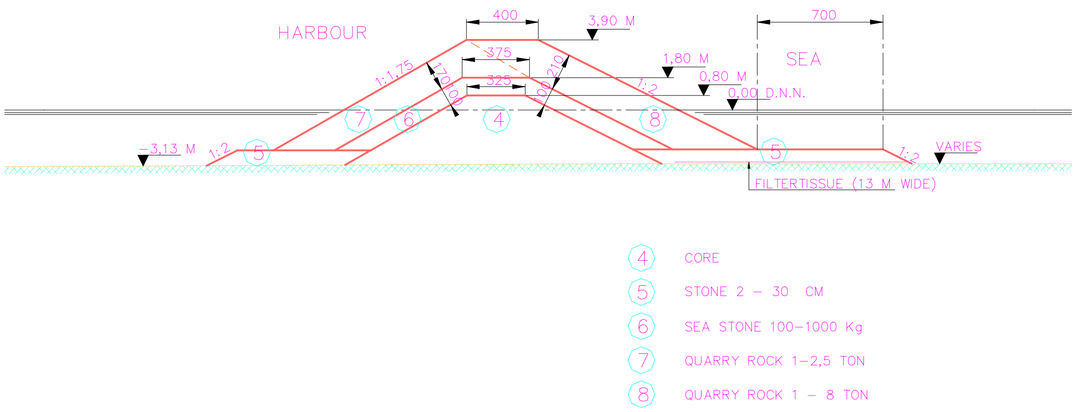


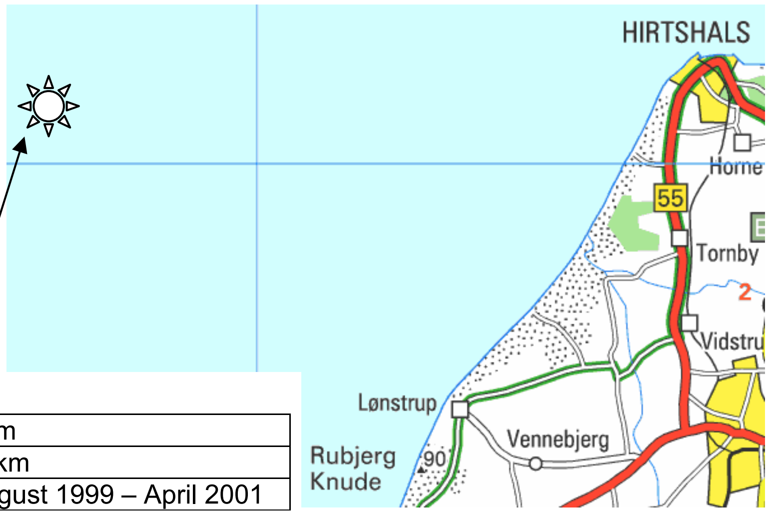
Figure B.20. Cross-section of breakwaters at Hirtshals Østmole. Measures in cm.

B.4.1 Water level variations

The same as at Lønstrup.

B.4.2 Wind and waves

Winds at Hirtshals are the same as at Lønstrup. Directional wave gauge recordings near Hirtshals exist in two locations; 1) A long data series from a location 68km from the coast at 166m's water dept, and 2) A short data series (less than 2 years) from a location 25 km from the coast at 25m's water depth. Only the wave recordings for the last location will be repeated here. The waves are grouped in 8 directions; N, NE, E, SE, S, SW, W and NW.



Wave gauge no 1041	
Water depth at wave gauge	25m
Distance to coast	25km
Period of measurements	August 1999 – April 2001

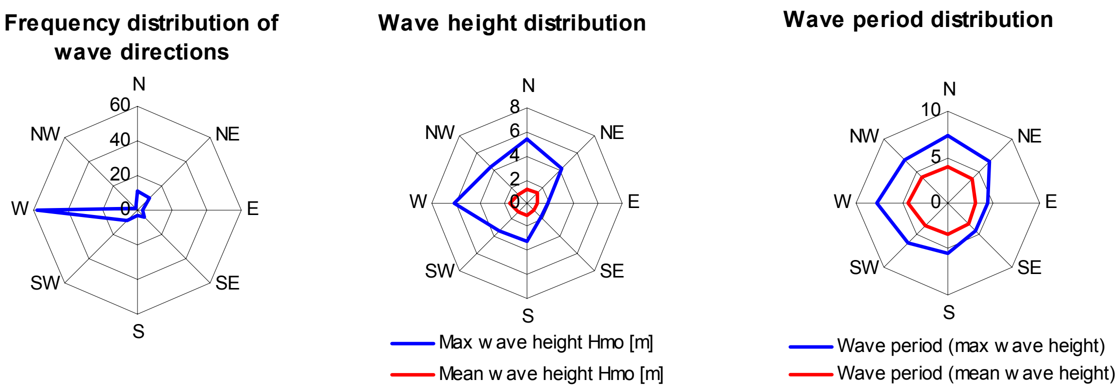


Figure B.21. Wave characteristics at Hirtshals.

It is seen from Figure B.21 (left) that 60% of the waves come from the West direction. The maximum recorded wave height is 6.1m (H_{m0} from the West, T=7.7 sec) and the largest mean wave height is 1.5m (H_{m0} from the west, T=4.4 sec). In average the mean wave height is 1.3m. The waves are measured 25km's from the coast in deep waters. Close to the breakwaters the waves are depth limited.

DHI has made a numerical simulation with an incoming wave Hs = 5.7 m in direction 290° (WNW). It is seen from Figure B.22 that just outside Østmolen the wave heights are approx. 0.25 m (in Eastern end) and approx. 0.75 m (where Nordmolen begins).

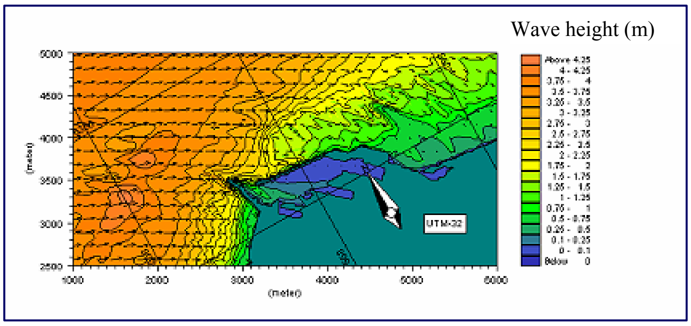


Figure B.22. Wave heights at Hirtshals Harbour. [1]

If the bottom topography is used to calculate the depth limited waves the typical significant wave height is 0.6·h = 0.6m (Western end/Nordmolen end) and 0.2 m (Eastern end). This seems to be roughly the same as DHI's numerical simulation. The highest wave at storm setup is estimated as 0.8·h = 2.0m (Western end) and 1.4 m (Eastern end).

B.4.3 Currents and sediments

Wind between N and SSW gives currents in E direction. Wind between NNE and S gives currents in W direction. However the current is generally from W to E. For information about general sediment transport, see Figure B.17 on page 118.

For an incoming wave Hs = 5.7 m in direction 290° (WNW, same as for Figure B.22) DHI has also calculated related currents, see Figure B.23.

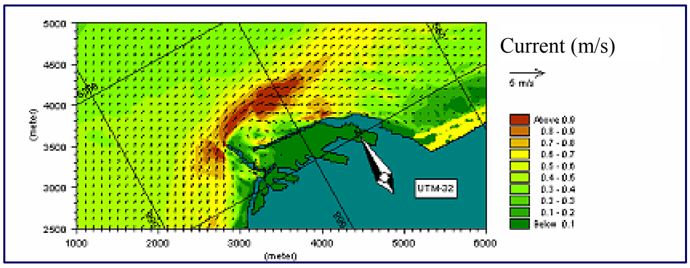


Figure B.23. Currents at Hirtshals Harbour. [1]

It is seen that the velocities near Østmolen are very low, approx. 0-0.4 m/sec.

In Figure B.12 it is seen that D_{n50} is approx. 0.2-0.3mm at Hirtshals.

[1] The figure is from a PowerPoint Presentation shown by Ida Brøker, DHI at a seminar about coastal protection in Lemvig, Denmark the 4th - 5th of May 2001.

B.4.4 Ecology

The ecological study of the harbour defence structures comprised intertidal surveys horizontally along Østmolen (The Eastern Pier) and subtidal surveys vertically of Dækmolen (The Western Pier). The harbour defence structures protect the rather large industrial harbour. Dækmolen is among the most exposed marine habitats of Denmark. The boulders are larger than on the other Danish sites studied here and include both concrete armour units and granite boulders. These are not uniformly distributed between depths or exposure forces. At Dækmolen, it is not uncommon that parts of the outer pier are broken loose and washed away during winter storms. The biota was studied on the seaward sides only and was carried out as described in Chapter B.2.4. Intertidal data was obtained from the horizontal transect along Østmolen by quantitative methods and subtidal data was obtained from five vertical transects along Dækmolen. Dækmolen show the largest difference in maximum depths of water, from the inner part near the beach (0 m) to the outer part at the harbour entrance (13 m).

Flora and fauna was present on both Østmolen and Dækmolen. However, the diversity was larger on the subtidal parts than on the intertidal parts.

B.5 Study site comparisons and summary

The data from each site are summarized in the following tables and figures.

B.5.1 Engineering assets

Based on Table B.6 to B.9 some of the major differences are summarized:

- Gaps between the breakwaters at Lønstrup are larger than at Skagen
- No nourishment at Hirtshals Østmole but nourishment at both Lønstrup and Skagen
- The water depth is generally low but at Hirtshals Østmole it is very low
- The armour layer stone size at Hirtshals Østmole is larger than at the other locations.

Table B.6. Geometry summary.

	$L_{segment}$ [m]	Gap [m]	Dist. [m]	Freeboard [m]	Width [m]	Depth [m]	Beach slope
Skagen	40	25	25	+1.0	3	1	1:100
Lønstrup	45	45	40	+1.3	2	1	1:150
Hirtshals	-	-	-	+3.9	-	0.3-1.0	Varies

Table B.7. Materials summary.

	Armour layer rock type	Armour layer stone size d_{n50}	Beach material	Beach material grain size
Skagen	Quarry rock	0.7m (Toe: 0.6m)	quartzite	0.25mm
Lønstrup	Quarry rock	0.8m (Toe: 0.55m)	quartzite	0.2mm
Hirtshals	Quarry rock	1.1m	quartzite	0.2-0.3mm

Table B.8. Nourishment summary.

	Regular sand fill behind breakwaters
Skagen	Yes
Lønstrup	Yes
Hirtshals	(No)

Table B.9. Hydrodynamics summary.

	Tidal range (typical)	Low water storm surge	High water storm surge	Highest Wave Height	Typical significant wave height	Wave direction	Current direction (from-to)
Skagen	0.3m	-0.9m	+1.4m	1.9m	0.6m	E-SE	SW-NE
Lønstrup	0.3m	-1.0m	+1.5m	2.0m	0.6m	W	S-N
Hirtshals	0.3m	-1.0m	+1.5m	1.4m * 2.0m **	0.2m * 0.6m **	NW	W-E

* In Eastern end of Østmolen ** In Western end of Østmolen

B.5.2 Ecology

In summary, the biota (flora and fauna) is sparse or absent on both the landward and seaward sides of the beach breakwaters. Biota diversity in number of species increases with water depth. In the intertidal zone the epibiota was 45 taxa (incl. 36 species). Subtidally the epibiota was more diverse and pelagic fauna including fish appears common, particularly at the deeper parts of the Grey Light House defence (between 2-4 m) and at Hirtshals Dækmole (between 2-13 m).

Table B.10. List of intertidal taxa indentified on LCS's in Northern Denmark. Names in bold indicate the most abundant species of algae and invertebrates found in intertidal habitats.

ALGAE:	INVERTEBRATES:
Enteromorpha spp.	*Dynamena pumila* (Linnaeus)
***Enteromorpha linza* (Linnaeus) J. Agardh**	Gastropods
Ulva lactuca Linnaeus	*Littorina littorea* (Linnaeus)
Cladophora spp.	***Littorina saxatilis* (Olivi)**
Hildenbrandia rubra (Sommerf.) Menegh.	***Littorina neritoides* (linnaeus)**
Porphyra umbilicalis (Linnaeus) C. Agardh	*Nucella lapillus* (Linnaeus)
***Mastocarpus stellatus* (Stackhouse) Guiry**	*Mytilus edulis* (SL>2 cm) Linnaeus
***Chondrus crispus* Stackhouse**	***Mytilus edulis* (SL<2cm) Linnaeus**
***Ceramium rubrum* (Huds.) C. Agardh**	Polychaetes (tubiferous)
Polysiphonia spp.	**Barnacles,** *Semibalanus balanoides* (Linnaeus)
Phaeophyta, filamentous.	***Idotea* spp.**
Furcellaria lumbricalis (Huds.) Lamour.	Amphipods
Fucus spiralis Linnaeus	*Carcinus maenas* (Linnaeus)
Fucus vesiculosus Linnaeus	***Electra pilosa* (Linnaeus)**
Fucus serratus Linnaeus	*Asterias rubens* Linnaeus
Ascophyllum nodosum (Linnaeus) Lejol.	
Sargassum muticum (Yendo) Fensholt	
Laminaria spp.	

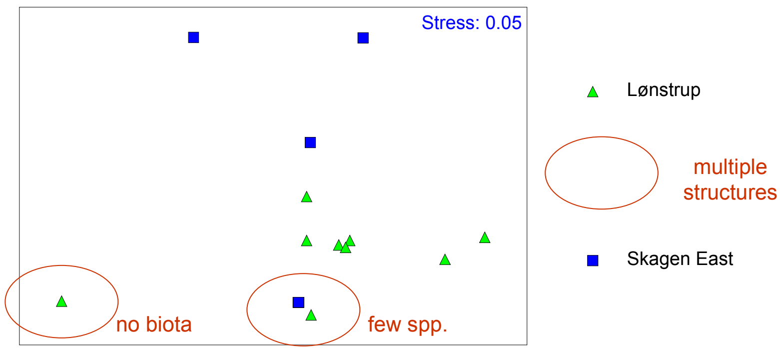


Figure B.24. The intertidal biota by the semi-quantitative measures on breakwaters plotted in MDS. The ANOSIM 1-way test shows clear difference of biota composition between the sites Skagen and Lønstrup.

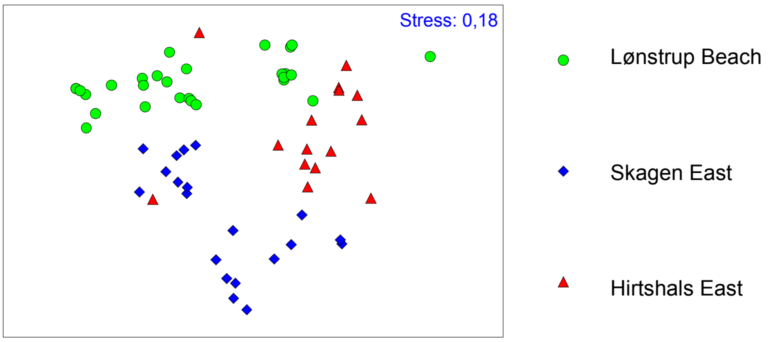


Figure B.25. The intertidal biota by the quantitative measures on defence structures plotted in MDS. The ANOSIM 1-way test show differences of biota composition between the sites Skagen, Lønstrup, and Hirtshals.

Figure B.26. Position of the subtidal transects at Hirtshals Harbour (Dækmolen) and Skagen Eastern Beach.

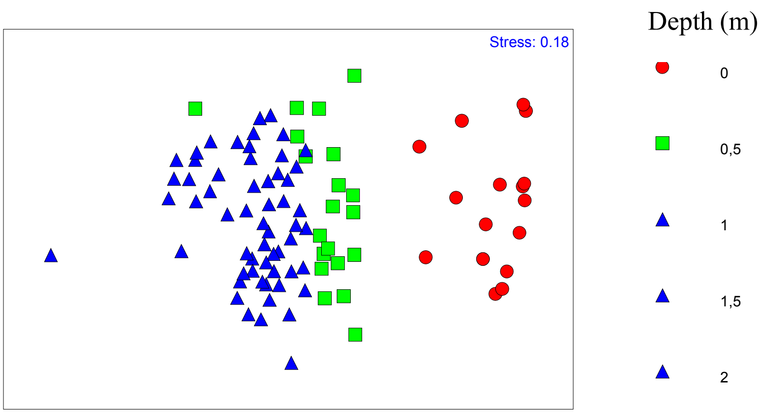


Figure B.27. The semi-quantitative data from the vertical transect survey as MDS-plot by depth. The ANOSIM 1-way test shows that the largest difference in biota composition is between the intertidal (0 m) and the subtidal (0.5 - 2 m) part of structures. The biota diversity increases with depths. Among the subtidal depths, 0.5 m differ from the greater depths, which are also more divers in species.

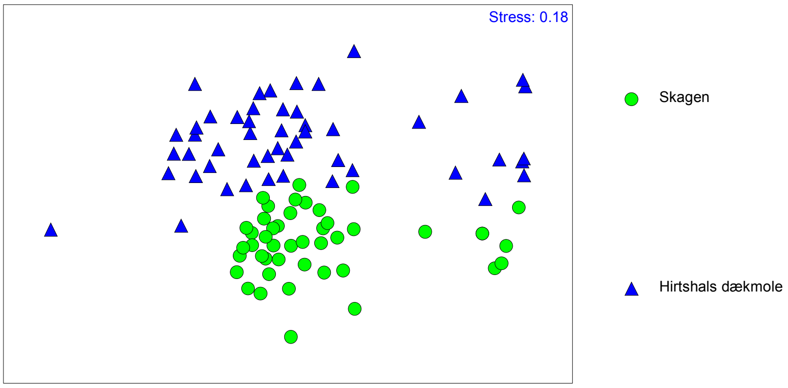


Figure B.28. The MDS-plot of the semiquantitative data from the vertical transect survey as MDS-plot by site. The ANOSIM 1-way test shows that the biota composition differs between the defence of the Grey Light House and the Hirtshals Harbour defence of Dækmolen.

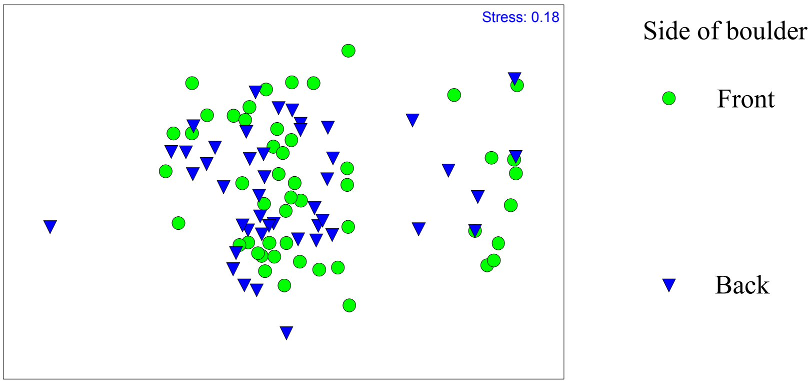


Figure B.29. The semi-quantitative data from the vertical transect survey as MDS-plot by seaward frontal and seaward shaded sides of boulder. The ANOSIM 1-way test shows that no significant difference between frontal and back (shaded) faces of the structures are detectable by this type of sampling.

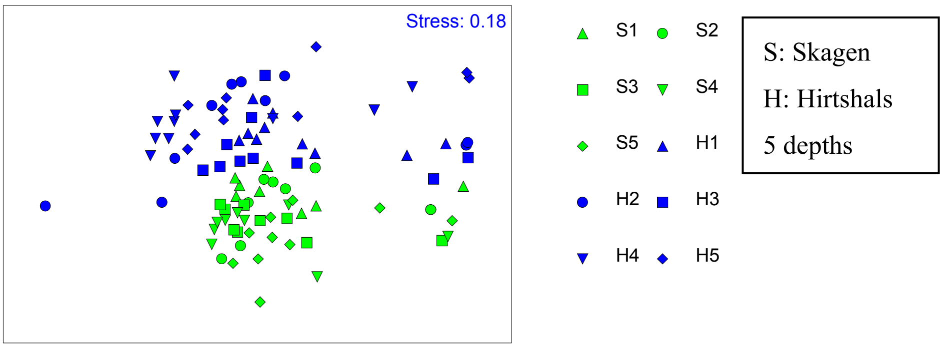


Figure B.30. The semi-quantitative data from the vertical transect survey as MDS-plot by transect. The ANOSIM 2-way crossed test shows that biota diversity differs significantly with both depth and transects.

Appendix C : 2D stability tests at AAU 2005

The 2D stability tests were carried out at Aalborg University during autumn 2005 in order to verify the rule of thumb. In the following details about the tests are given in order to supplement the descriptions in Chapter 5.3.

C.1 Wave channel description

The flume is 25 m long 1.5 m wide and 1 m deep. Maximum water depth is approx. 75cm. The flume is equipped with a piston type wave generator with a stroke length of ±70cm. The wave generator is controlled by a PC controlled DHI Servo Amplifier. The generation software is AWASYS. AWASYS is an active absorption system, which was used to generate irregular waves. AWASYS is developed by the laboratory. For further information see http://www.hydrosoft.civil.aau.dk/index.htm.

Pictures of the layout are given in Figure C.1 and drawings with dimensions and further descriptions are given in Chapter 5.3.

Figure C.1. Pictures from 2D stability tests at Aalborg University 2005. Wave gauges are not shown.

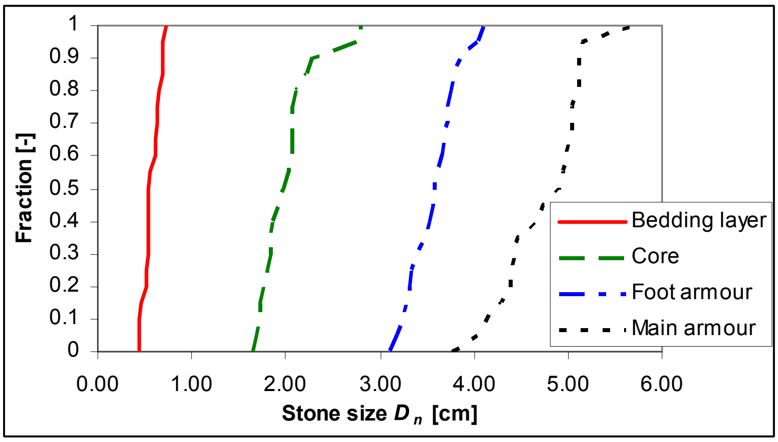
C.2 Materials

All materials in the structure including the beach consisted of quarry rock with the typical mass density of ρ_s = 2650 kg/m^3. Measurements of were performed in order to verify the mass density. As fresh water with mass density ρ_w = 1000 kg/m^3 was used in the flume the relative density for the stones were $\Delta = \rho_s / \rho_w - 1 = 1.65$.

The grading and the nominal diameters of the materials were measured from samples of 21 stones of each type; see Table C.1 and Figure C.2. As the sample sizes were relatively small there is a small statistical uncertainty on the given characteristic parameters.

Table C.1. ***Grading of trunk materials in the 2D tests at Aalborg University 2005.***

Fraction	Main armour		Foot armour		Core		Bedding layer	
	W_s (g)	D_n (cm)	W_s (g)	D_n (cm)	W_s (g)	D_n (cm)	W_s (g)	D_n (cm)
0.00	142	3.77	79	3.10	12.0	1.65	0.22	0.44
0.05	175	4.04	86	3.19	12.7	1.69	0.24	0.45
0.10	188	4.14	91	3.25	13.4	1.72	0.24	0.45
0.15	**210**	**4.30**	**93**	**3.27**	**13.6**	**1.72**	**0.27**	**0.47**
0.20	224	4.39	96	3.31	14.4	1.76	0.37	0.52
0.25	226	4.40	99	3.34	15.6	1.81	0.38	0.52
0.30	231	4.43	104	3.40	16.3	1.83	0.39	0.53
0.35	235	4.46	110	3.46	16.6	1.84	0.42	0.54
0.40	274	4.69	117	3.53	17.0	1.86	0.43	0.55
0.45	281	4.73	121	3.57	18.9	1.92	0.43	0.55
0.50	**315**	**4.92**	**123**	**3.59**	**20.6**	**1.98**	**0.43**	**0.55**
0.55	321	4.95	124	3.60	22.5	2.04	0.47	0.56
0.60	328	4.98	131	3.67	23.2	2.06	0.60	0.61
0.65	337	5.03	132	3.68	23.3	2.06	0.63	0.62
0.70	340	5.04	136	3.72	23.5	2.07	0.65	0.63
0.75	341	5.05	136	3.72	23.8	2.08	0.65	0.63
0.80	354	5.11	140	3.75	24.9	2.11	0.72	0.65
0.85	**354**	**5.11**	**142**	**3.77**	**29.3**	**2.23**	**0.86**	**0.69**
0.90	356	5.12	151	3.85	31.8	2.29	0.89	0.70
0.95	364	5.16	175	4.04	56.7	2.78	0.89	0.70
1.00	492	5.70	183	4.10	57.5	2.79	1.06	0.74



Stone type	D_{n50} [cm]	D_{n85}/D_{n15}
Main armour	4.92	1.19
Foot armour	3.59	1.15
Core	1.98	1.29
Bedding layer	0.55	1.47

Figure C.2. ***Grading of trunk materials in the 2D tests at Aalborg University 2005.***

C.3 Sequence of operations including building the structure

Initially the breakwater was built of natural coloured stones. Bedding layer and core materials were manually dumped from buckets and distributed by hand. Foot and armour layer stones were carefully placed manually in order to ensure a compact structure, as if the stones were placed by crane in real applications.

The channel was filled with water and some wave situations were run in order to make visual inspection of wave breaking characteristics and to allow for some initial settlements of the structure. In order to ensure similar wave breaking characteristics in all wave runs no matter of the water depth it was decided to do all tests with the same peak period T_p = 1.8 s, and with the same value of H_s/h at the wave paddle as input to the wave generation software.

The flume was drained and the surface of the structure was spray painted with red colour. Damage to the structure, i.e. a stone moved from its initial position, would thereby cause a natural coloured spot in the surface of the structure.

Wave gauges, cameras and lights were mounted, and the flume was filled until the water level reached the crest level. The structure height was measured to 17 cm, which was exactly the target height. The flume was drained until the lowest water level was reached, and the tests were initiated.

After the tests were finished the structure height was measured again in order to ensure that no settlements had taken place during the tests. The structure height was confirmed, i.e. no settlements had taken place.

C.4 Measurements

Arrays of three wave gauges were used in order to extract incoming waves, reflected waves and noise from the measurements. Three wave gauges (numbered 1, 2 and 3 in Figure C.3) were placed in front of the structure allowing for determination of the incoming wave and reflection characteristics. Three wave gauges were placed behind the structure (numbered 4, 5 and 6 in Figure C.3) allowing for determination of set-up, wave transmission and reflection from the beach. Wave gauge 4 was placed one metre behind the structure in order to avoid measuring splashes of overtopping breaking waves.

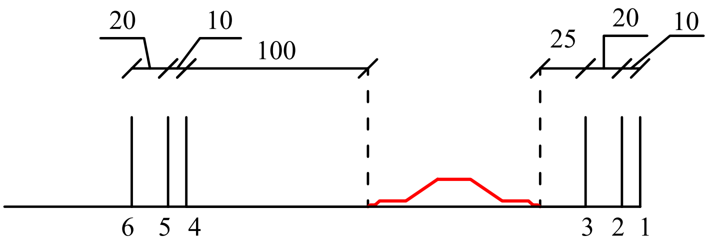


Figure C.3. Position of the six wave gauges. Measures in cm.

Damage was measured by taking digital photos before and after each test. Digital video was taken to visualize wave breaking and impacts to the structure.

C.5 Results

The structure stayed stable and no damage took place except for in one test. Damage to the armour layer happened in test 6 corresponding to freeboard R_c/H_c = -0.35, see Table C.2. In general the armour layer was most prone to damage for slightly submerged conditions. The toe stayed stable during all tests, but visually rocking stones were identified at the lowest water depths (test 1 and 2) corresponding to a water level at around the level of the toe.

Table C.2. Damage to the trunk during the 2D tests at Aalborg University 2005.

Test	h [cm]	R_c [cm]	R_c/H_c	Wave transmision	Damage
1	4.0	13.0	0.76	None	No damage but some stones rocking in the toe
2	9.0	8.0	0.47	No overtopping but some wave penetration	No damage but some stones rocking in the toe
3	13.0	4.0	0.24	Some overtopping and wave penetration	None
4	17.0	0.0	0.00	A lot of overtopping	None
5	20.0	-3.0	-0.18	A lot of overtopping	No damage but some stones rocking in the crest
6	23.0	-6.0	-0.35	A lot of overtopping	One stone in damage to the armour and many (app. 5) stones rocking in crest
7	26.0	-9.0	-0.53	A lot of overtopping	No damage but a little rocking in the crest
8	30.0	-13.0	-0.76	A lot of overtopping	None
9	34.0	-17.0	-1.00	A lot of overtopping	None

As no active control of the set-up behind the structure was performed, a relatively large set-up compared to the freeboard developed during the experiments. This set-up created in some situations unrealistic return flows in the opposite direction of the waves over the crest. Due to wave grouping and the small distance between the beach and the structure some large waves were able to create a relatively large setup, which during low wave action led to large return flows. Visually changes in wave breaking characteristics were noticed, and the feeling that the existence of the return flow would lead to more damaging conditions (i.e. less stable structure). As no counter modifications to the set-up were performed, it is believed that the results of the experiments possibly are slightly on the safe side. The measurements from the wave gauges were used to find the average set-up, see Table C.3. From the table it is seen that small negative values of the set-up are found at the location of wave gauge in front of the breakwaters (at wave gauge 1, 2 and 3), meaning that the water level is decreased giving a little set-down in front of the structure. Behind the structure (at wave gauge 4, 5 and 6) the water level is increased giving a set-up of up to approximately 1.45 cm in test number 4. The average value of the set-up for wave gauge 4, 5 and 6 (right column in Table C.3) is in the following used to define the set-up behind the structure S_u.

Table C.3. Measurements of mean set-up. Values are in cm.

		Wave gauge position							
		1	2	3	4	5	6	Average of 1, 2, 3	Average of 4, 5, 6
Test number	1	-	-	-	-	-	-	-	-
	2	-0.06	-0.03	-0.07	0.20	0.21	0.21	-0.05	0.21
	3	-0.18	-0.16	-0.19	0.52	0.50	0.51	-0.18	0.51
	4	-0.40	-0.36	-0.39	1.49	1.42	1.43	-0.39	1.45
	5	-0.29	-0.27	-0.34	1.03	0.99	0.99	-0.30	1.00
	6	-0.36	-0.29	-0.36	0.61	0.61	0.62	-0.34	0.61
	7	-0.29	-0.24	-0.28	0.31	0.32	0.30	-0.27	0.31
	8	-0.19	-0.22	-0.26	0.06	0.10	0.04	-0.22	0.07
	9	-0.15	-0.11	-0.09	0.04	0.03	-0.01	-0.12	0.02

It is seen from Table C.4 and Figure C.4 that the largest set-up corresponds to zero freeboard (test no. 4). Further it is observed that zero freeboard gives a much higher value of the relative set-up S_u/h than emerging or submerging conditions. The measured surface elevation is filtered with an average filter in order to investigate the time variation in the set-up. The time variation in the filtered surface elevation for zero-freeboard condition is shown in Figure C.5. From the figure it is seen that the filtered curves follows the black dashed curves with only minor deviations. This indicates that only small changes in the set-up takes place in time due to wave grouping effects.

The wave steepness from measurements at wave gauge 3 is calculated by $s_{0p} = H_{s,3}/L_{0p}$, where $L_{0p} = gT_p^2 / 2\pi = 5.1$ m for $T_p = 1.8$ s. Damage happened in test 6 corresponding to $R_c/H_c = -0.35$. In this test the wave steepness was $s_{0p} = 0.021$, which is typical for real wave conditions. In test 6 the relative set-up was $S_u/h = 0.027$, i.e. only slightly less than 3% of the water depth, indicating that return-flows in this tests was small.

Table C.4. Details about test conditions and wave measurements.

Test	h [cm]	R_c [cm]	$H_{s,3}$ [cm]	S_u [cm]	R_c/H_c	$H_{s,3}/h$	s_{0p}	S_u/h
1	4.0	13.0	2.0	-	0.76	0.51	0.004	-
2	9.0	8.0	4.5	0.21	0.47	0.50	0.009	0.023
3	13.0	4.0	6.1	0.51	0.24	0.47	0.012	0.039
4	17.0	0.0	7.8	1.45	0.00	0.46	0.015	0.085
5	20.0	-3.0	9.3	1.00	-0.18	0.46	0.018	0.050
6	23.0	-6.0	10.4	0.61	-0.35	0.45	0.021	0.027
7	26.0	-9.0	11.3	0.31	-0.53	0.43	0.022	0.012
8	30.0	-13.0	12.9	0.07	-0.76	0.43	0.025	0.002
9	34.0	-17.0	15.2	0.02	-1.00	0.45	0.030	0.001

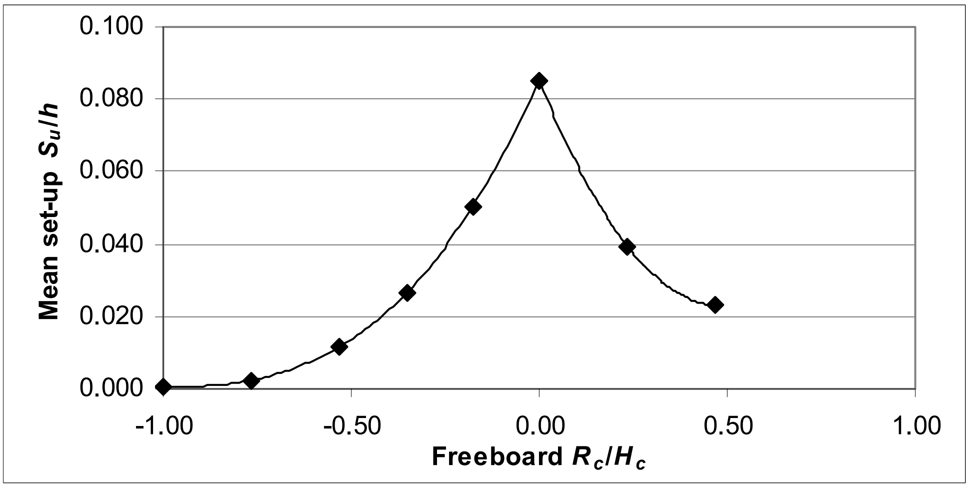


Figure C.4. Average set-up behind the trunk as function of the freeboard. The line indicates the trend of the data.

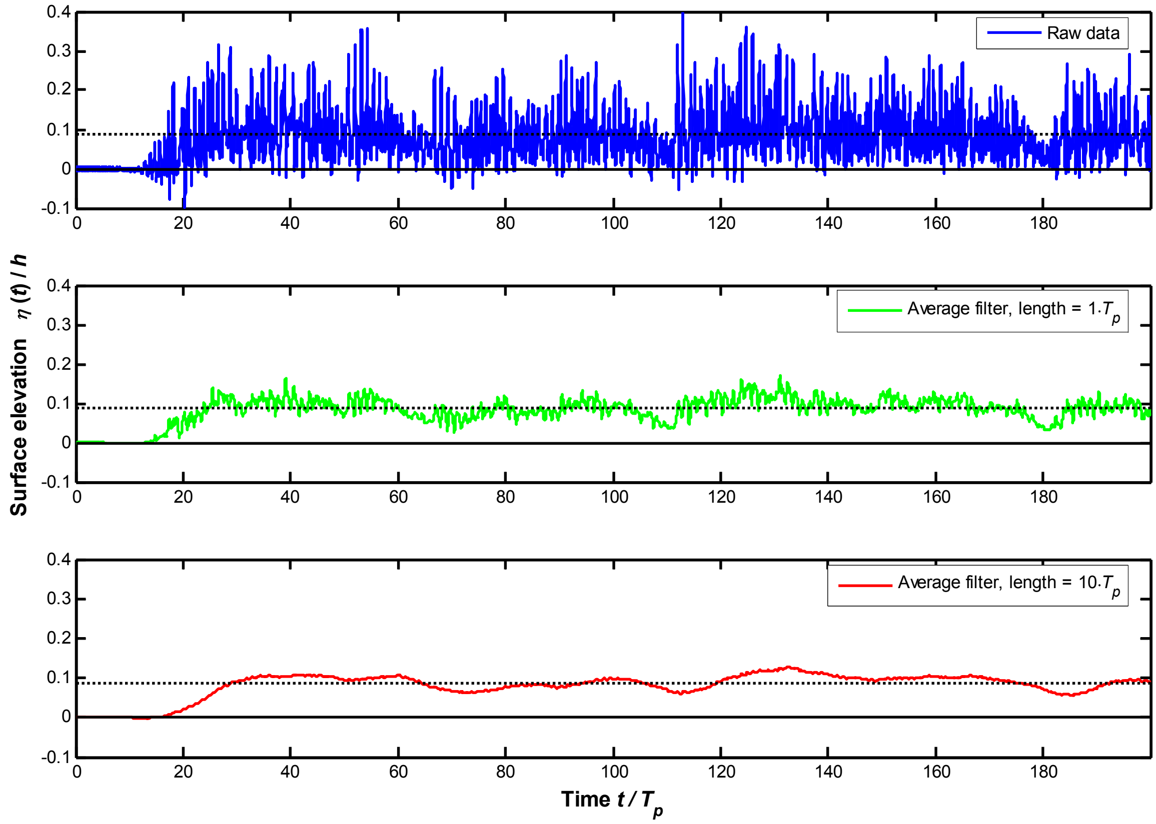
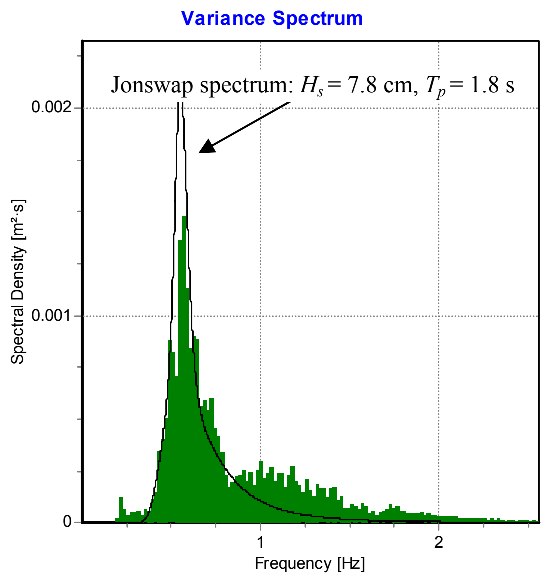


Figure C.5. Time variation in set-up. Measurements from wave gauge 4 for zero freeboard R_c = 0. Dashed line is the average set-up for the whole timeserie of length $t = 811T_p$. No waves at t = 0 and fully developed waves at $t \approx 30T_p$.

As the waves were depth limited wave breaking influenced the spectral shape and the wave height distribution. From Figure C.6 (left) it is seen that wave energy at the peak frequency (0.56 Hz) is decreased and distributed by wave breaking around the double peak frequency (1.11 Hz). From Figure C.6 (right) it is seen that the wave heights does not follow the Rayleigh distribution, as very high waves cannot exist at the low water depth.



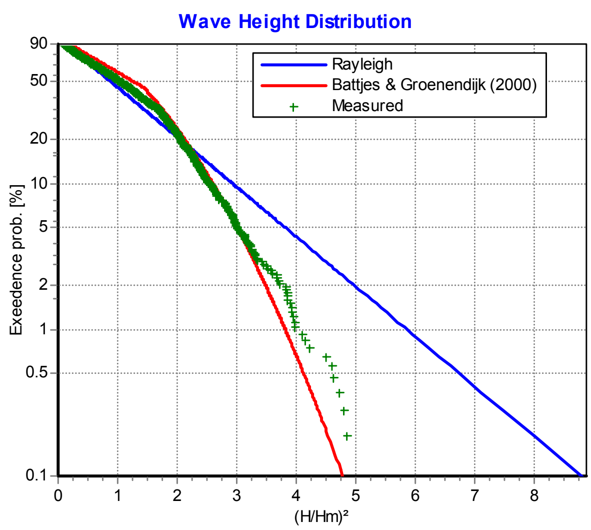


Figure C.6. Example of wave spectrum and wave height distribution. Measurements from wave gauge 1 for tests with zero freeboard.

Appendix D : 3D Stability tests at AAU 2002

The stability tests were carried out in the short-crested wave basin at Aalborg University in Denmark during the summer 2002. The text given in this chapter is a revision of a report published through the DELOS project, see Kramer *et al.* (2003).

D.1 Wave basin layout

The basin used for the DELOS tests is 12 m long, 18 m wide and 1.0 m deep as shown in Figure D.1. The paddle system is a snake-front piston type composed of 25 actuators with a stroke length of 1.2 m, enabling generation of short-crested waves. The wave generation software used for controlling the paddle system is Profwaco developed by the laboratory. Regular and irregular short crested waves with peak periods up to approximately a maximum of 3 seconds can be generated with acceptable result. Oblique 2D and 3D waves can be generated.

A fixed seabed made of concrete was used in all tests. The absorbing sidewalls were made of crates (121x121cm, 70 cm deep) filled with sea stones with D_{n50} of app. 5 cm, see Figure D.1 (right). The areas outside the crates were left empty in all the tests. The beach was made of quarry rock with D_{n50} = 1.5 cm. The beach was made of quarry rock with D_{n50} = 1.5 cm.

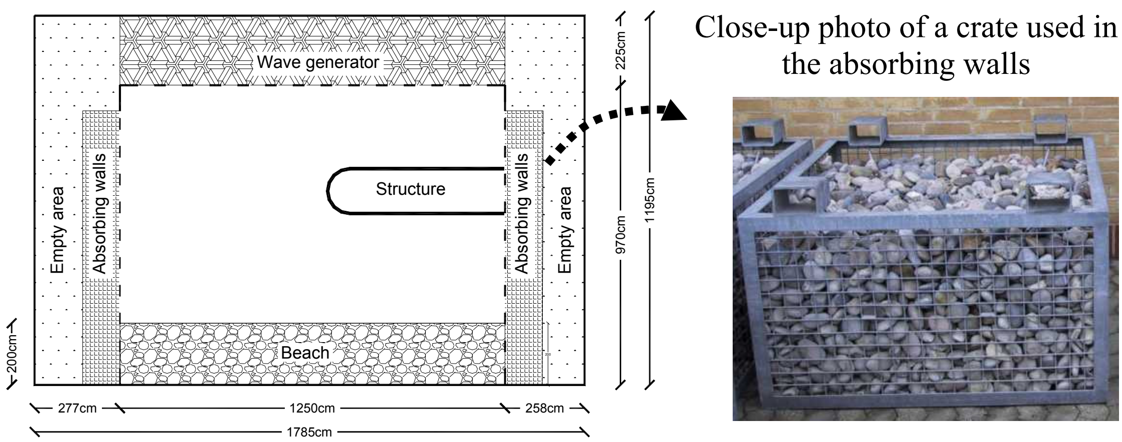


Figure D.1. Wave basin layout with position of structure.

D.2 Materials

The rubble stones used for armour layers in the test sections (Type A) were quarry rock with mass density ρ_s = 2650 kg/m^3. In order to get well graded armour material in the test sections and to avoid very flat or long stones all the stones were carefully selected manually one-by-one.

All Type A stones were spread out on the floor, mixed, and a random sample containing 169 stones was extracted. Each individual stone was weighed, and the length (X), width (Y) and height (Z) was measured. The length was taken as the longest dimension, and the height as the shortest dimension. Figure D.2 (right) shows that 80% of the Type A stones have X/Z < 2, and that all stones have X/Z < 3. This means that Type A contains no flat or long stones.

The Type B stones contained some flat and long stones but was only used for the dummy trunk section shown in Figure 5.7. The Type C stones used for the main dummy part of the trunk contained sizes large enough to avoid displacements during the tests.

From each type of material a sample was taken, and the nominal diameter D_n of each individual stone was calculated from the weight W and the mass density ρ_s. Stone types A, B and C were narrow graded, cf. Table D.1 and Figure D.3. For the core was used more wide graded stones (Type D), cf. Table D.1 and Figure D.3.

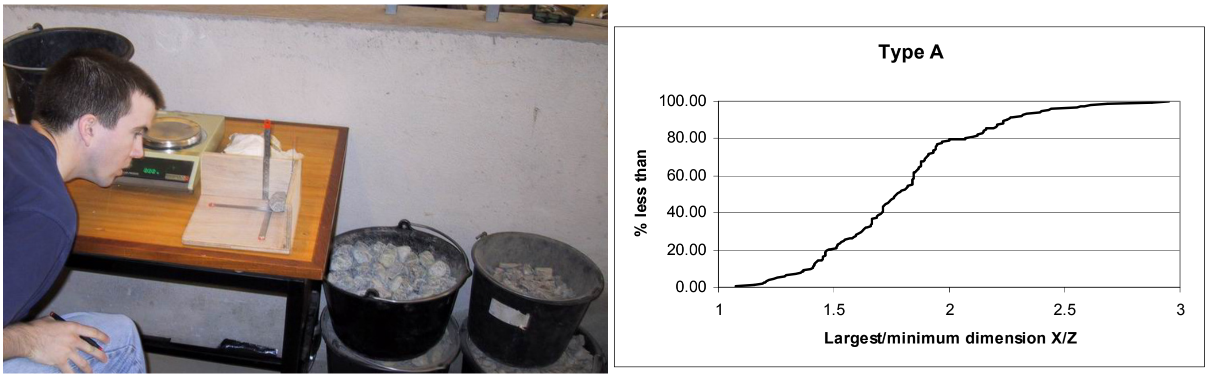


Figure D.2. Left: Manual measurements in lab. Right: Curve describing the length/height-ratio.

Table D.1. Nominal diameters for materials.

	D_{n50} [cm]	D_{n85} [cm]	D_{n15} [cm]	D_{n85}/D_{n15} -
Type A	3.25	3.60	3.01	1.20
Type B	3.07	3.43	2.68	1.28
Type C	4.74	5.24	4.32	1.21
Type D	1.44	1.83	1.11	1.64

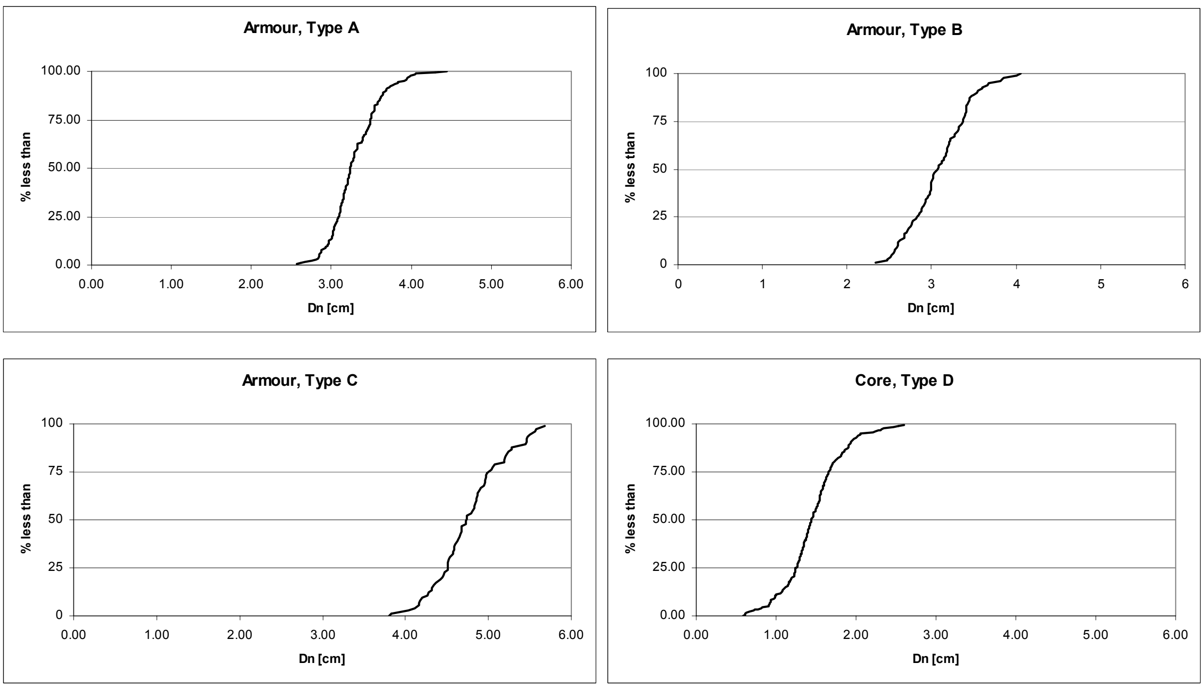


Figure D.3. Grading of materials.

The porosity n for armour Type A and core Type D was calculated in the following way. A sample of stones with bulk volume V was weighed without water in the pores W_s. The corresponding volume of the voids V_v was measured by adding water to the sample. The porosity was calculated as follows:

Porosity (directly by volume of voids) $= V_v/V$

Porosity (by weight of stone) $= (V - W_s/\rho_s)/V$

A sample size was chosen such that the two estimates gave the same porosity. For Type A $V = 14$ litres was chosen, and for Type D $V = 2$ litres was chosen. The result was $n_{(\text{Type A})} = 0.44$ and $n_{(\text{Type D})} = 0.43$.

To identify damage and to follow each individual stone's path Type A stones were painted in different colours. The stones were immersed in thin paint for a short time and spread out on the floor to dry. In that way only a thin layer of paint was added and the surface roughness of the material was only slightly altered. Seven colour codings were used: Red (R), green (G), blue (B), black (K), white (W), yellow (Y) and no colour (N).

D.2.1 Building of the breakwater model

Without water in the basin the position of the breakwater was marked with chalk on the seabed. Core material was spread out and a templet constructed in wood was used to ensure correct height and slopes. Armour material was then spread out randomly on the core by pouring the stones from buckets. A templet was used to ensure target slopes and thickness of the armour layer. Manual adjustment of the profile was necessary.

The basin was filled with water such that the water depth was equal to target zero freeboard. The crest height was then given a final adjustment by moving and adding stones such that a precise freeboard was obtained.

D.3 Wave conditions

In all tests a Jonswap spectra with peak enhancement factor 3.3 and a spreading parameter $S_\theta = 50$ was used as input to the wave generator.

D.3.1 Calibration tests

Initially 34 calibration tests without the model structure in place were performed with irregular 3D waves. The purpose was to ensure that correct wave conditions were reproduced, and to investigate the influence of the sloping foreshore on the wave breaking. Two deepwater wave steepness' $s_{0p} = 0.02$ and $s_{0p} = 0.04$ were tested with four to five wave heights (ranging from no wave breaking to a lot of wave breaking). Four water depths were investigated corresponding to the depths used in the subsequent tests. A wave gauge array consisting of 5 individual gauges was positioned where the roundhead of the breakwater was to be placed in the subsequent experiments. It was confirmed that in case of non breaking waves the wave generator produced a wave spectrum very close to the target. In general most waves started to break on the top edge of the foreshore slope. When a lot of wave breaking took place (more than 50% of the waves were breaking) a significant wave height to water depth ratio of $H_s/h \cong 0.5$ was observed at the investigated location. In the actual tests with the structure present the waves were depth limited. Wave breaking was therefore important and is described in more detail in chapter D.4.

D.3.2 Actual tests

The target length of each series was 1000 waves. A test block was defined by fixed water level, wave direction, wave steepness, and spreading. In each test block the significant wave height was increased in steps until severe damage was observed. It was attempted to get four tests in each block. However, this was not possible in all blocks due to the progress of the damage. Target conditions were therefore continuous adjusted according to target damage during a tests block. After each block the breakwater was rebuilt. The following describes the procedure applied in a test block.

- Built/rebuilt the structure
- Fix water level, wave direction, steepness and spreading
- Perform test with 1000 waves with small wave height
- Measure damage
- Increase significant wave height and run 1000 waves
- Measure damage
- ...continue to increase the wave height and measure damage until severe damage was observed

Table D.2 Target conditions for the narrow-crest structure

Test no.	Test name	Time [sec]	β [°]	Crest width [m]	Free-board [m]	Wave steepness	Hs deep [m]	Tp deep [s]
1	Test001	1140 (19min)	0	0.1	0.05	0.02	0.05	1.27
2	Test002	1380 (23min)	0	0.1	0.05	0.02	0.075	1.55
3	Test003	1500 (25min)	0	0.1	0.05	0.02	0.1	1.79
4	Test004	1680 (28min)	0	0.1	0.05	0.02	0.125	2.00
5	Test005	840 (14min)	0	0.1	0.05	0.04	0.05	0.90
6	Test006	1020 (17min)	0	0.1	0.05	0.04	0.075	1.10
7	Test007	1140 (19min)	0	0.1	0.05	0.04	0.1	1.27
8	Test008	1260 (21min)	0	0.1	0.05	0.04	0.125	1.42
9	Test009	1140 (19min)	0	0.1	0	0.02	0.05	1.27
10	Test010	1380 (23min)	0	0.1	0	0.02	0.075	1.55
11	Test011	1500 (25min)	0	0.1	0	0.02	0.1	1.79
12	Test012	1680 (28min)	0	0.1	0	0.02	0.125	2.00
13	Test013	840 (14min)	0	0.1	0	0.04	0.05	0.90
14	Test014	1020 (17min)	0	0.1	0	0.04	0.075	1.10
15	Test015	1140 (19min)	0	0.1	0	0.04	0.1	1.27
16	Test016	1260 (21min)	0	0.1	0	0.04	0.125	1.42
17	Test017	1380 (23min)	0	0.1	0	0.04	0.15	1.55
18	Test018	1380 (23min)	0	0.1	-0.05	0.02	0.075	1.55
19	Test019	1500 (25min)	0	0.1	-0.05	0.02	0.1	1.79
20	Test020	1680 (28min)	0	0.1	-0.05	0.02	0.125	2.00
21	Test021	1740 (29min)	0	0.1	-0.05	0.02	0.15	2.19
22	Test022	1860 (31min)	0	0.1	-0.05	0.02	0.175	2.37
23	Test023	1140 (19min)	0	0.1	-0.05	0.04	0.1	1.27
24	Test024	1260 (21min)	0	0.1	-0.05	0.04	0.125	1.42
25	Test025	1380 (23min)	0	0.1	-0.05	0.04	0.15	1.55

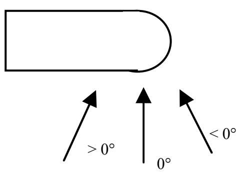
Test no.	Test name	Time [sec]	β [°]	Crest width [m]	Free-board [m]	Wave steepness	Hs deep [m]	Tp deep [s]
26	Test026	1500 (25min)	0	0.1	-0.05	0.04	0.175	1.67
27	Test027	1560 (26min)	0	0.1	-0.05	0.04	0.2	1.79
28	Test028	1680 (28min)	0	0.1	-0.1	0.02	0.125	2.00
29	Test029	1740 (29min)	0	0.1	-0.1	0.02	0.15	2.19
30	Test030	1860 (31min)	0	0.1	-0.1	0.02	0.175	2.37
31	Test031	1920 (32min)	0	0.1	-0.1	0.02	0.2	2.53
32	Test032	1260 (21min)	0	0.1	-0.1	0.04	0.125	1.42
33	Test033	1380 (23min)	0	0.1	-0.1	0.04	0.15	1.55
34	Test034	1500 (25min)	0	0.1	-0.1	0.04	0.175	1.67
35	Test035	1560 (26min)	0	0.1	-0.1	0.04	0.2	1.79
36	Test036	1620 (27min)	0	0.1	-0.1	0.04	0.225	1.90

Table D.3 Target conditions for the wide-crest structure

Test no.	Test name	Time [sec]	β [°]	Crest width [m]	Free-board [m]	Wave steepness	H_s deep [m]	T_p deep [s]
37	Test037	1140 (19min)	0	0.25	0.05	0.02	0.05	1.27
38	Test038	1380 (23min)	0	0.25	0.05	0.02	0.075	1.55
39	Test039	1500 (25min)	0	0.25	0.05	0.02	0.1	1.79
40	Test040	1680 (28min)	0	0.25	0.05	0.02	0.125	2.00
41	Test041	1140 (19min)	-20	0.25	0.05	0.02	0.05	1.27
42	Test042	1380 (23min)	-20	0.25	0.05	0.02	0.075	1.55
43	Test043	1500 (25min)	-20	0.25	0.05	0.02	0.1	1.79
44	Test044	1680 (28min)	-20	0.25	0.05	0.02	0.125	2.00
45	Test045	1140 (19min)	-10	0.25	0.05	0.02	0.05	1.27
46	Test046	1380 (23min)	-10	0.25	0.05	0.02	0.075	1.55
47	Test047	1500 (25min)	-10	0.25	0.05	0.02	0.1	1.79
48	Test048	1680 (28min)	-10	0.25	0.05	0.02	0.125	2.00
49	Test049	1140 (19min)	+10	0.25	0.05	0.02	0.05	1.27
50	Test050	1380 (23min)	+10	0.25	0.05	0.02	0.075	1.55
51	Test051	1500 (25min)	+10	0.25	0.05	0.02	0.1	1.79
52	Test052	1140 (19min)	+20	0.25	0.05	0.02	0.05	1.27
53	Test053	1380 (23min)	+20	0.25	0.05	0.02	0.075	1.55
54	Test054	1500 (25min)	+20	0.25	0.05	0.02	0.1	1.79
55	Test055	1680 (28min)	+20	0.25	0.05	0.02	0.125	2.00
56	Test056	1140 (19min)	-30	0.25	0.05	0.02	0.05	1.27
57	Test057	1380 (23min)	-30	0.25	0.05	0.02	0.075	1.55
58	Test058	1500 (25min)	-30	0.25	0.05	0.02	0.1	1.79
59	Test059	1680 (28min)	-30	0.25	0.05	0.02	0.125	2.00
60	Test060	1140 (19min)	0	0.25	0	0.02	0.05	1.27
61	Test061	1380 (23min)	0	0.25	0	0.02	0.075	1.55
62	Test062	1500 (25min)	0	0.25	0	0.02	0.1	1.79
63	Test063	1680 (28min)	0	0.25	0	0.02	0.125	2.00
64	Test064	1500 (25min)	0	0.25	-0.05	0.02	0.1	1.79
65	Test065	1680 (28min)	0	0.25	-0.05	0.02	0.125	2.00
66	Test066	1740 (29min)	0	0.25	-0.05	0.02	0.15	2.19
67	Test067	1680 (28min)	0	0.25	-0.1	0.02	0.125	2.00
68	Test068	1740 (29min)	0	0.25	-0.1	0.02	0.15	2.19
69	Test069	1860 (31min)	0	0.25	-0.1	0.02	0.175	2.37

Note on wave direction:

- > 0° : Most of the back head is sheltered from direct wave attack
- 0° : Normal incidence waves perpendicular to structure
- < 0° : A large part of the head is exposed to direct wave attack



D.4 Measurements

Three kinds of measurements were performed:

- Waves were recorded continuous during the tests.
- Wave breaking was described from visual observations.
- Damage in terms of displacement of stones was measured after each test by use of digital photos. Damage was classified in categories. Digital video recordings were taken during a few tests of special interest.

D.4.1 Wave recordings

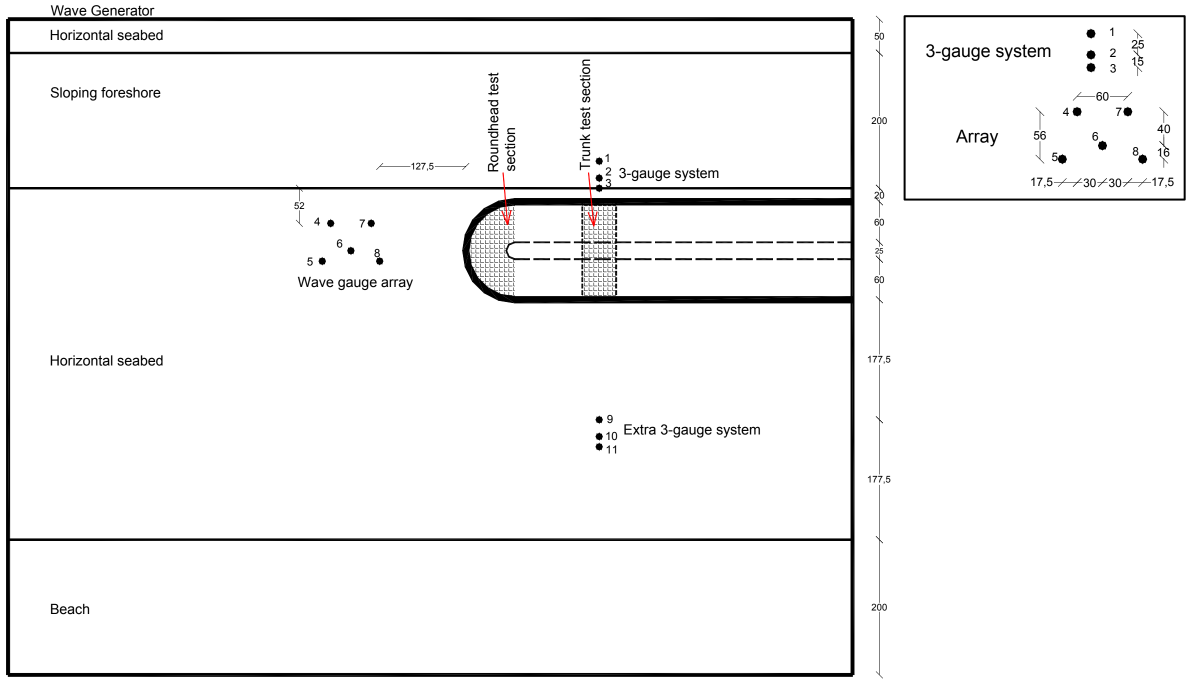


Figure D.4. Position of wave gauges. Measures in cm.

Recordings by an array of five wave gauges can be used to estimate incoming and reflected wave spectra. At the position of the array almost 1.5 metres from the roundhead the influence of the roundhead (reflection and diffraction) on the incoming waves is believed to be negligible. However, the trunk reflects some wave energy which is re-reflected by the paddles. Therefore the waves in front of the trunk might in reality be slightly higher (and/or more wave breaking) than at the array. Measurements from the 3-gauge system were performed to quantify that effect. In some wave situations a lot of waves were expected to be breaking in front of the structure, and the measurements were therefore possibly very dependent on the gauge position. The 3-gauge system was placed close to the structure with distances 60, 35 and 20cm to the foot of the trunk. As 3D waves were generated these gauges cannot be used in a traditional reflection analysis.

The purpose of the measurements from the extra 3-gauge system (located on the leeward side of the structure) was to be able to compare with possible future numerical wave calculations. It was not the intention to use these measurements in the stability considerations.

Data files were stored in ASCII text format, one file for each test, with test number as file-name. Each column in a file corresponds to a wave gauge such that data in column no 1 are sampled from wave gauge no 1, etc. In that way every file has 11 columns.

Measured surface elevation data is in cm generally with zero at still water level. However all wave gauges might not be precisely adjusted to zero at still water level. Positive surface elevation indicates a wave crest passing.

All data were sampled at 20Hz from start of wave generation.

D.4.2 Wave breaking

Wave breaking on the foreshore slope or on/over the structure was carefully monitored. In general the following was observed during a test block of four tests with increasing significant wave height.

1) Smallest waves that gave no damage:
   - Gentle lapping of waves against trunk crest only
   - Very few waves (<10%) were breaking over trunk crest

2) Second smallest waves that in some part of the structure moved a few stones:
   - Some waves were breaking (approx 50%) over the trunk crest
   - Very few waves were breaking on top edge of foreshore slope

3) Second largest waves that in some part of the structure gave significant damage:
   - Most waves were breaking over the trunk crest
   - Few waves were breaking on top edge of the foreshore slope

4) Largest waves that in most part of the structure gave severe damage:
   - Almost all waves were breaking over the trunk crest
   - A lot of the waves were breaking on the top edge of the foreshore slope
   - Very few waves were breaking on foreshore slope before reaching the top edge

In some cases the wave breaking was concentrated at the roundhead forming a jet of water and air slamming down on the top part of leeward head (between blue and green stone shown subsequent on Figure D.6). This led frequently to severe damage of the leeward part of the roundhead.

D.4.3 Measurement of damage

Four pictures were taken in between each test. Three pictures were taken of the roundhead and one of the trunk. Picture 1 shows the seaward side of the roundhead, picture 2 the roundhead seen from the gap, picture 3 the leeward side of the roundhead, and picture 4 the trunk seen from a position vertically above the centre of the trunk section. Digital video (📼) of selected tests were recorded from the gap.

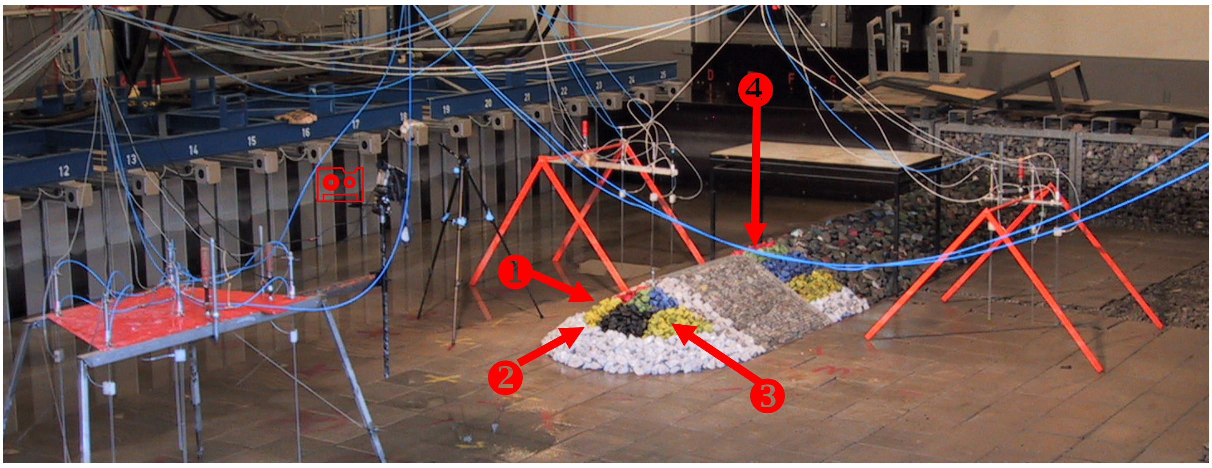
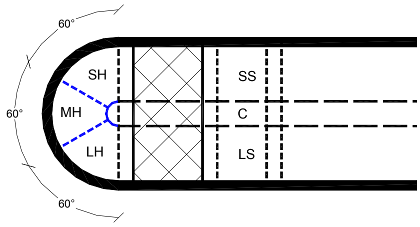


Figure D.5. Position of pictures for measurement of damage.

The colouring of the roundhead was split in three sections of 60° each. The three sections were called: Seaward Head (SH), Middle Head (MH) and Leeward Head (LH). The trunk was split in three parts called: Seaward Slope (SS), Crest (C), and Leeward Slope (LS).



The precise colouring is shown in the following figures.

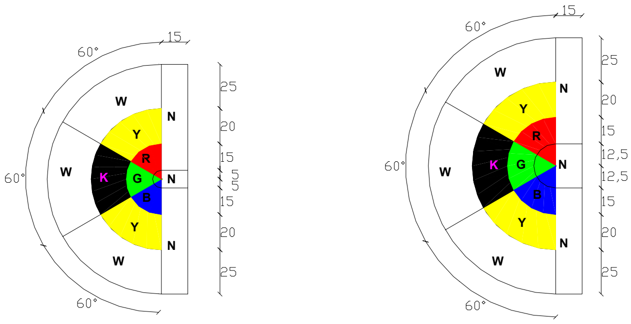


Figure D.6. Colouring of roundhead. Left: Narrow structure. Right: Wide structure. Measures in cm.

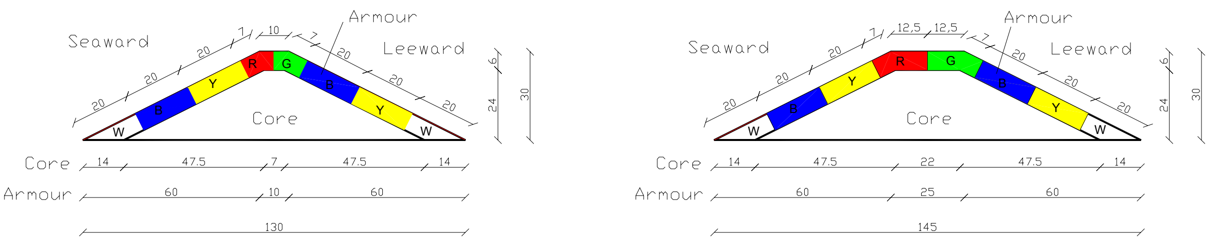


Figure D.7. Colouring of trunk. Left: Narrow structure. Right: Wide structure. Measures in cm.

The digital pictures were imported into a program for photo viewing, and by switching back and forth between pictures before and after a test it was possible to follow the path of every individual stones and to count the number of stones that moved in that particular test. A stone was defined to have moved, when it moved more than one D_{n50} away from its original position. The following example, Figure D.8 and Figure D.9, shows how to count the number of stones. After test number 19 no stones had moved from the original position in the roundhead. After test number 20 four stones had moved.

Figure D.8. Picture from position 2. Left: Before test 20. Right: After test 20.

The number of stones that have moved is easily counted from Figure D.8:

- Seaward Head (SH): 3R (three red stones have moved)
- Middle Head (MH): 1G (one green stone has moved)
- Leeward head (LH): 0 (no movement)

Figure D.9. Tracking the stone movements.

When more than approximately 20 stones moved, the actual number had to be roughly estimated. This was generally only the case when the structure was heavily damaged or close to total destruction (filter layer often exposed to direct wave attack).

The degree of damage was also assessed visually and categorized as follows (according to definitions by Losada *et al*., 1986):

- ND: No damage (maybe one or two loose stones starts rotating)
- ID: Initiation of damage (a few stones starts to move)
- IR: Iribarren damage (big holes in the outer armour layer, but the filter layer is not visible).
- D: Destruction (filter layer is exposed to direct wave attack)

The example on Figure D.8 and Figure D.9 is for the roundhead categorized as: ID for seaward head and ND for middle head and leeward head. The categorisation is described in detail in Chapter 3.

D.5 Video recordings and CD file contents

A CD containing the source data and other information about the stability tests is available. The CD is categorized in the following folders:

- “Data” contains recorded wave data in ASCII text format. Wave data are compressed in the file “Data.zip”. The wave data files contain surface elevation measured in cm at 20Hz.
- “Documents” contains documents describing the tests plus databanks with analysed waves, analysed damage and stone gradings. Documents are in Microsoft Word 2002 format and databanks in Microsoft Excel 2002 format.
- “Drawings” contains AutoDesk AutoCAD 2002 drawings of detailed layout and cross-sections in the tests.
- “Pictures” contains jpeg pictures for damage estimation and some general pictures from the experiments.

Digital video of selected tests were recorded from the gap (for position of camera see on Figure D.5), see Table C.4. The video is stored on mini DV-tapes and are kept at:

Hydraulics & Coastal Engineering Laboratory
Aalborg University
Department of Civil Engineering
Sohngaardsholmsvej 57
9000 Aalborg
Denmark

Table D.4. Available video recordings.

Tape number	Test number
1	4
2	12
3	17
4	22
5	40
6	54 and 55

To get a CD or to borrow the tapes or get copies of selected sequences please contact Morten Kramer (i5mkr@civil.aau.dk) from Aalborg University.

D.6 Target and actual wave conditions

In general target and actual significant wave heights were approximately the same also for the breaking waves. The definition of wave height to be used in the stability considerations is fundamental; therefore the wave heights are described in detail.

D.6.1 Wave heights

The structure was expected to produce slightly higher waves in front of the trunk than what was measured with the wave gauge array, see Figure D.4. For the array H_{mo} was calculated with directional wave analysis by the Bayesian Direct Method (Hashimoto and Kobune, 1987). H_s was calculated by time domain analysis for the three individual wave gauges in the 3-gauge system. These H_s’s were expected to be larger than the actual incoming H_s due to wave reflection and re-reflections. On Figure D.10 the wave heights from the 3-gauge system are compared to the results from the array. "Hs1" in Figure D.10 corresponds to H_s for gauge number 1 (gauge farthest from structure, see Figure D.4 for position of wave gauges), etc.

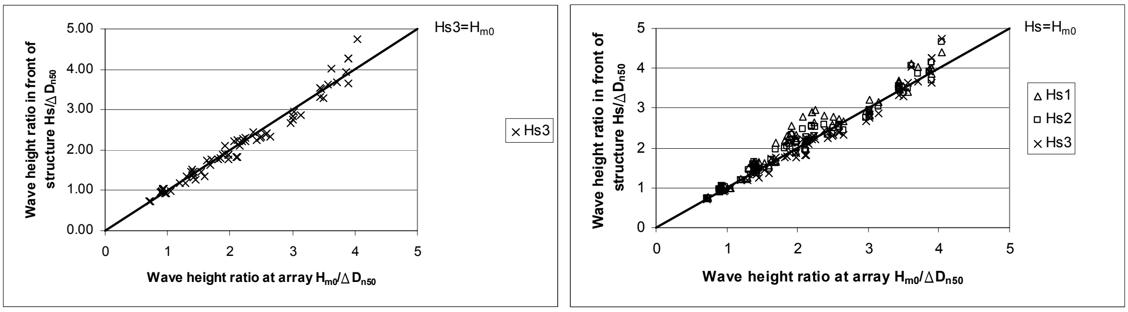


Figure D.10. Comparison of waves in front of structure (left: H_s for gauge 3, right: H_s for the 3 individual gauges) with waves at array (H_{m0}). *Hs3* is closest to structure.

From Figure D.10 (left) it is seen that points follow the line $Hs3 = H_{m0}$, H_s from gauge 3 in the 3-gauge system is therefore approximately equal to H_{m0} from the array. This indicates that the influence of reflected and re-reflected waves between the structure and the paddles is marginal.

At Figure D.10 right is seen that the waves closest to the structure (*Hs3*) are a bit smaller than the average, and that the waves farthest to the structure (*Hs1*) are a bit larger than the average. In most tests the largest waves were depth limited. Wave gauge number 1 and 2 were located on the foreshore slope at a larger water depth than the structure. As larger water depth allows larger waves it is obvious that *Hs1* should be larger than *Hs3*. H_s, $H_{2\%}$ (wave height with probability of exceedance 2%) and $H_{1\%}$ were calculated for the gauges in the 3-gauge system, and the average values for all tests were found as given in Table D.5. In Table D.5 it is seen that the wave height ratio based on H_s decreases from $H_s/\Delta D_{n50} = 2.31$ (at gauge no. 1) to 2.20 (at gauge no. 2) to 2.12 (at gauge no. 3). This corresponds to an average significant wave height 4% larger at gauge 2 compared to gauge 3, and a 9% larger significant wave height at gauge 1 compared to gauge 3. The same decrease in wave height is found for the average $H_{2\%}$ and $H_{1\%}$.

Table D.5. Average wave height ratios in front of structure.

	Gauge 1	Gauge 2	Gauge 3
$H_s/\Delta D_{n50}$	2.31	2.20	2.12
$H_{2\%}/\Delta D_{n50}$	3.03	2.89	2.76
$H_{1\%}/\Delta D_{n50}$	3.20	3.04	2.91

It is clear that the wave height distribution changes as the waves approach the structure. This is shown further in the following example. Test number 4 was a test with large breaking waves, which lead to severe damage of the structure in all sections.

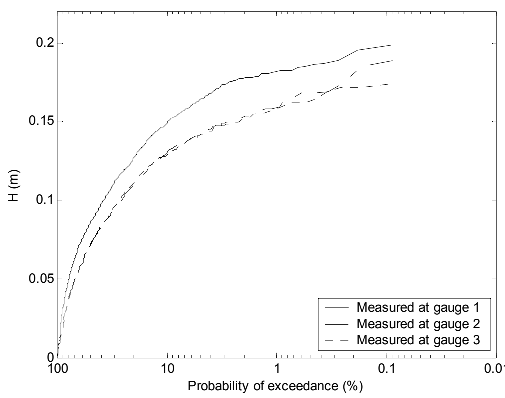


Figure D.11. Measured wave height distribution for the 3-gauge system, test number 4.

From Figure D.11 it is clear, that especially the highest waves are higher at gauge number 1 than at gauge 3. In test number 4 the water depth at gauge 3 was 0.25 m and at gauge 4 it was 0.266 m, i.e. 6.5 % larger water depth at gauge 1. The $H_{2\%}$ was measured to 0.178 m at gauge 1 and 0.152 m at gauge 3, i.e. a 17 % larger wave height at gauge 1.

The change in wave height distribution is investigated in more detail in Figure D.12. The measured wave height distribution is compared to the Rayleigh distribution and to the point model proposed by Battjes and Gronendijk (2000). Battjes and Gronendijks model is developed for wave height distributions on shallow foreshores, and it takes account for water depth and foreshore slope.

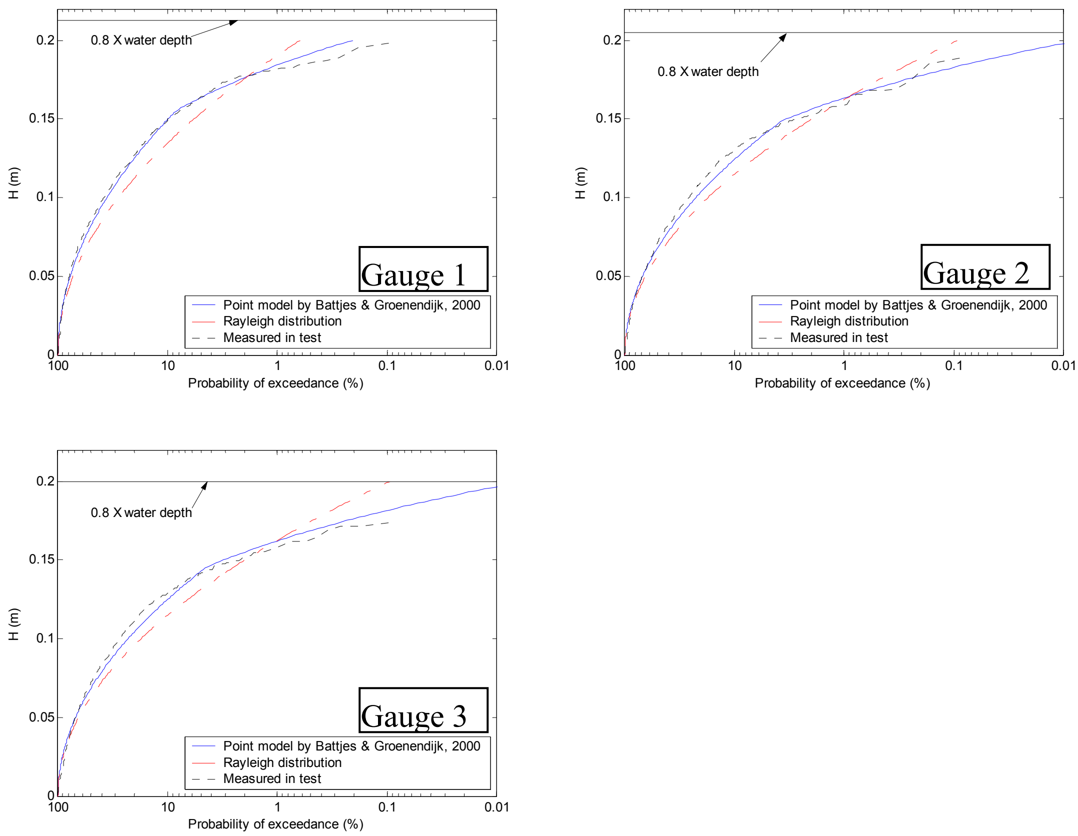


Figure D.12. Measured wave height distribution compared to calculated, test number 4.

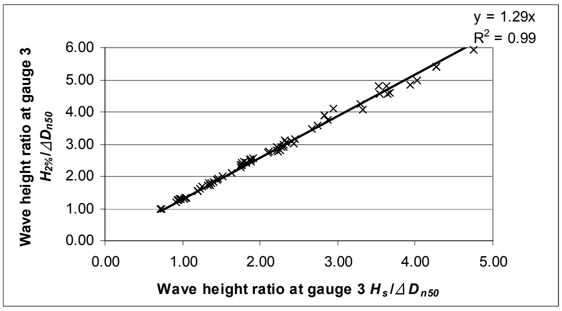
From Figure D.12 it is seen that the measured wave height distribution deviates from the Rayleigh distribution. Further it is seen that the point model fits the measured distribution from test number 4 outstandingly well. As LCS's are built in shallow waters the point model seems to be a good tool in describing wave height distributions at a given location.

Wave heights, concluding remarks

Wave height measurements from gauge 3 are appropriate in describing H_s for all wave directions. It is therefore chosen to use a stability number based on measurements from gauge 3 in the stability considerations subsequent.

Because the highest waves lead to damage of the structure, and because the waves are depth limited leading to changes in wave height distribution, it could be reasonable to use a more infrequent wave height than the significant wave height in the damage descriptions. However, the experimental results can be converted by a multiplication factor, which is clarified in the following. From Table D.5 the average measured $H_{2\%}$ and $H_{1\%}$ at gauge 3 is 30 % and 37 %

larger than H_s respectively. In Figure D.13 measured H_s's in all tests are compared to $H_{2\%}$ and $H_{1\%}$. The measured relation between $H_{1\%}$ or $H_{2\%}$ and H_s is constant, but it differs from the Rayleigh distribution. According to the Rayleigh distribution $H_{2\%} = 1.49 \cdot H_s$ and $H_{1\%} = 1.51 \cdot H_s$.



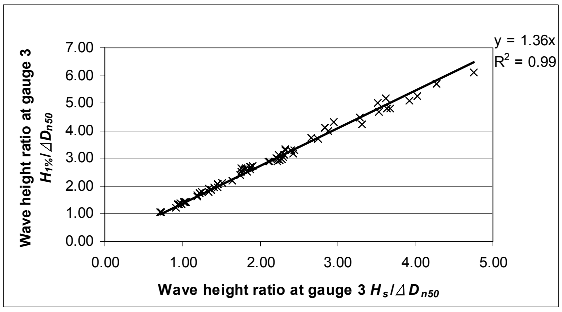


Figure D.13. Wave height ratios at gauge 3 and linear fit by use of $H_{2\%}$ (left) and $H_{1\%}$ (right).

It is seen that the plotted values on Figure D.13 fits the straight lines very well. In the following it is chosen to use Hs in the damage description. In case it is needed to make a damage description based on $H_{2\%}$ or $H_{1\%}$ Figure D.13 can be used for conversion. Wave heights with other exceedance probability are available in the Excel databank on the CD (see chapter D.5).

D.6.2 Peak period

Target and actual peak periods were in all cases approximately the same, also for cases with wave breaking.

D.6.3 Wave steepness

In the main part of the tests the target deepwater wave steepness was $H_{s,deep}/L_{0p} = 0.02$, and in the remaining part of the tests the target deepwater wave steepness was $H_{s,deep}/L_{0p} = 0.04$. For all the tests the actual wave steepness's defined by $s_{0p} = H_s/L_{0p}$ are calculated and plotted in Figure D.14. Measurements from wave gauge 3 are used to define H_s.

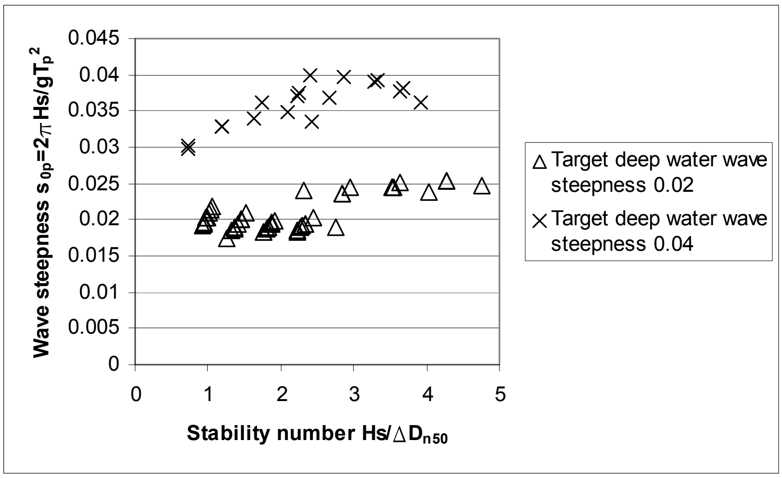


Figure D.14. Target and actual wave steepness's in all tests.

The average for all tests with target deepwater wave steepness 0.02 and 0.04 is $s_{0p} = 0.020$ and $s_{0p} = 0.035$ respectively. On Figure D.14 it is seen that s_{0p} is slightly increasing for higher stability numbers. However, in all tests the wave steepness's are close to the average values $s_{0p} = 0.02$ and $s_{0p} = 0.035$.

D.6.4 Number of waves

The target number of waves was 1000. The average numbers of waves for all tests were:

Gauge 1:	1012
Gauge 2:	1031
Gauge 3:	1037
Average:	1027

The actual number of waves was found as an average from the 3-gauge system. In all tests except for two the actual number of waves was 1000 ± 10%, however in 72% of the tests the number of waves was 1000 ± 5%.

The actual number of waves is considered to be in agreement with the target.

D.6.5 Main incoming wave direction

Normal incidence waves was defined the angle 0° (wave direction perpendicular to structure). Analysis showed that only two cases of normal incident waves were outside 0° ± 3°. The difference is only considered to be due to the statistical uncertainty in the analysis. Oblique waves with obliquity up to ± 20° were also produced correct. In one test block (test no 56 to 59) it was attempted to generate -30° oblique waves. Analysis showed that the actual main direction was only -23°. Test no 56-59 should therefore only be used with care, see Chapter D.6.6 for more detail.

For wave directions less than 0° (when a large part of the head was exposed to direct wave attack) the waves tend to get trapped between the structure, and the paddles and sidewall causing slightly larger waves in front of the structure than at the array. It is therefore important that wave heights from the 3-gauge system are used in the damage description, especially in case of oblique waves.

D.6.6 Spreading of incoming waves

In 86 % of the cases the standard deviation on the wave direction was in the range 9° to 15° (corresponding to S_θ-values in Mitsuyasu spreading function S_θ= 34 to S_θ= 109). Wave situations with the largest significant wave heights had the largest spreading and wave situations with the lowest significant wave heights had the lowest spreading. In average the standard deviation was 12.1° (corresponding to S_θ= 55). The input to the wave generator was S_θ = 50. As e.g. refraction and wave breaking will change the spreading, the actual measured conditions are considered to be in agreement with the target conditions.

Example of 3D wave spectra

It is chosen to show results from tests number 40 and 59 on the wide-crest structure. Tests 40 and 59 were tests with the largest tested wave heights at the lowest water depth. During the testing the wave breaking was described with the words *"A lot of the waves break on top edge of foreshore slope, almost all waves break over trunk crest"*. These tests led to severe damage in all sections of the structure.

In both tests the freeboard was R_c = +0.05 m (emerged crest), and the wave steepness corresponded to 0.02. In test number 40 waves with main direction head-on were generated, and in test number 59 it was attempted to generate -30° waves (a large part of the head exposed to direct wave attack). In Table D.6 the target wave is specified by the input to the wave generator. H_{m0}, spreading and main direction β are measured from the array. The number of waves is from gauge number 3.

Table D.6. Wave conditions in test number 40 and 59 at the wave gauge array.

Setup			Target wave			Measured wave			
Test	Water depth [m]	Freeboard [m]	H_s [m]	T_p [sec]	β [°]	H_{m0} [m]	Number of waves	Spreading [°]	β [°]
40	0.25	+0.05	0.125	2.0	0	0.114	1073	16	1
59	- ‖ -	- ‖ -	- ‖ -	- ‖ -	-30	0.100	1129	15	-30

Directional wave analysis by the Bayesian Direct Method (Hashimoto and Kobune, 1987) leads to the polar plot in Figure D.15. In Figure D.15 high energy content is marked with red colour meaning, and with blue colour meaning low energy. The direction of wave propagation 0:360° is shown along the circumference, and the frequency is shown as the radii from origo.

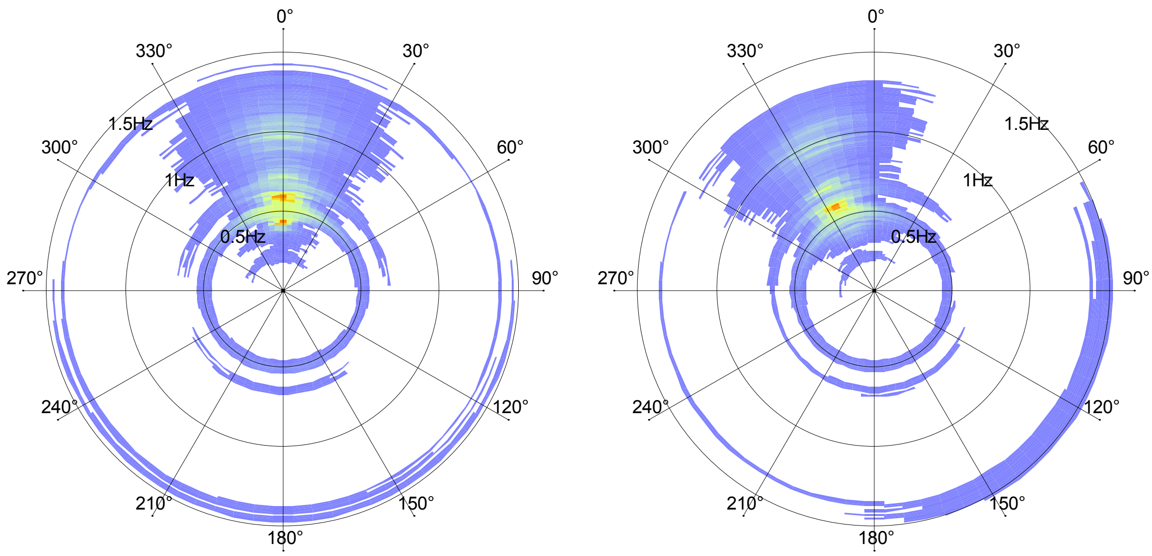


Figure D.15. 3D wave spectra for test number 40 (left) and 59 (right).

In Figure D.15 the effect of wave breaking is identified as a secondary peak at the double peak frequency, i.e. at 1 Hz. Due to oblique wave direction the waves in test no. 59 travel a longer distance in shallow water before hitting the structure and the wave gauges. Consequently wave breaking becomes more pronounced in test no. 59, and H_{m0} decreases. In Table D.6 it is seen that H_{m0} in test number 59 is 0.100 m, and in test number 40 it is 0.114 m.

In Figure D.15 (left) it is seen that a small amount of energy is present in the range 90° to 270°. This is due to reflections from the beach. The reflection is in average (for all tests) less than 15 % (reflected wave height compared to incoming wave height), largest for the largest waves in the deepest water. The reflection from the beach is generally very low, and the influence on the wave climate in front of the structure is therefore marginal.

Waves with main direction -30° were generated in test no. 59, but when the waves reached the array the main direction was only about -20° oblique. As the waves travel into shallow water refraction will change the obliquity and force the wave orthogonals to become more parallel to the foreshore and structure. For this reason the spreading of the oblique waves decreases slightly. Results from tests in series with main wave direction -30° are omitted in the following.

D.7 Stability under actual wave conditions

The freeboard and the wave height are the most important parameters in describing the stability of the structural sections. The normalized freeboard R_c/D_{n50} has therefore been used as one primary parameter, and the wave height ratio or stability number $N_s = H_s/\Delta D_{n50}$ as another primary parameter. As explained in Chapter D.6.1 measurements from wave gauge 3 are used to define H_s. Usually only marginal damage is accepted when designing a structure. Therefore it is chosen to investigate and compare results only for the category ID (Initiation of Damage).

D.7.1 Stability related to wave steepness and structural section

The investigation of the influence of wave steepness on stability was performed only for the narrow structure for normal incidence waves (main direction perpendicular to structure). For each section of the breakwater the results are compared, see Figure D.16. From the graphs it is seen that the data for $s_{0p} = 0.02$ and $s_{0p} = 0.035$ are fairly close in case of Initiation of Damage. However, the series with $s_{0p} = 0.02$ (long waves) tend to give slightly more damage than series with $s_{0p} = 0.035$ (short waves). This means the structure is more stable for $s_{0p} = 0.035$. It is also clear that the data fits the regression lines reasonably well.

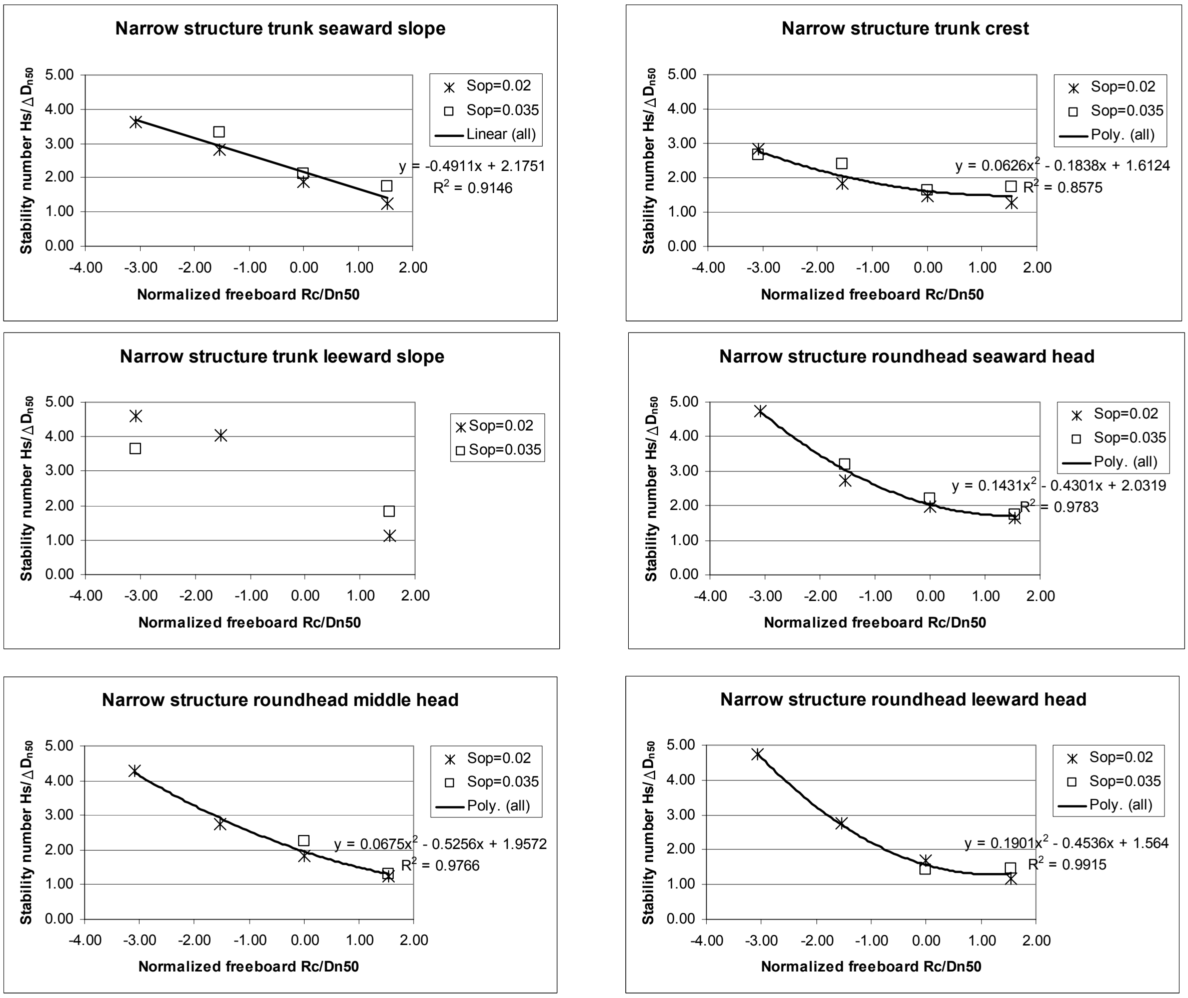


Figure D.16. Stability of narrow structure sections. Influence of wave steepness. Initiation of damage.

Further, the regression lines for different sections of the breakwater are compared in Figure D.17. For the trunk it is seen that the crest is the least stable section and that the leeward slope is the most stable part. For the roundhead the leeward head is the least stable part, and the stability of the middle head and seaward head is approximately the same.

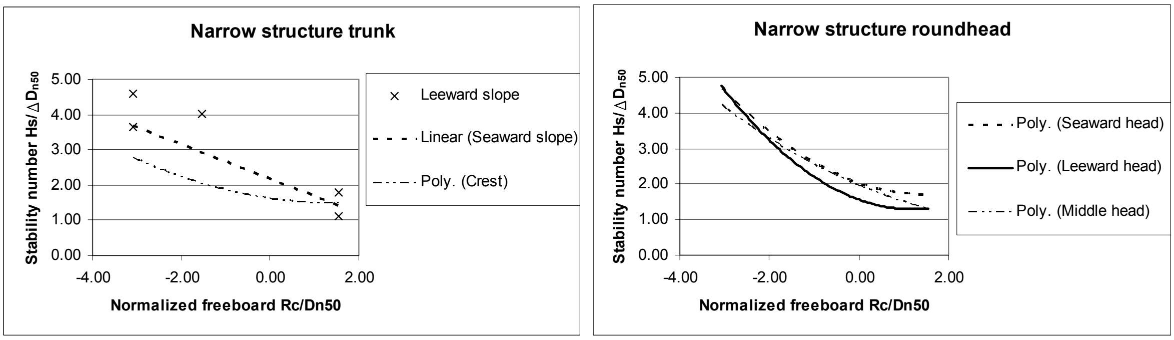


Figure D.17. Stability of the narrow structure. Trunk (left) and roundhead (right). Initiation of damage.

The stability of the head is further compared to the stability of the crest in Figure D.18. It is seen that the trunk crest is the least stable part under submerged conditions, and that for zero or emerged conditions the leeward head is the least stable.

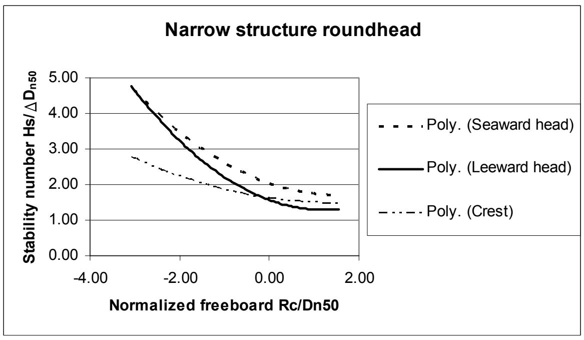


Figure D.18. Stability of narrow structure sections. Initiation of damage.

D.7.2 Stability related to crest width

Two structures with crest widths 0.1 m and 0.25 m (equal to $3D_{n50}$ for the narrow crest and $8D_{n50}$ for the wide crest) were tested in normal incidence waves with $s_{0p} = 0.02$. In Figure D.19 the stability of the structures is compared. No significant clear difference in response can be identified for the tested crest widths. Further it is seen that the influence of crest width is small.

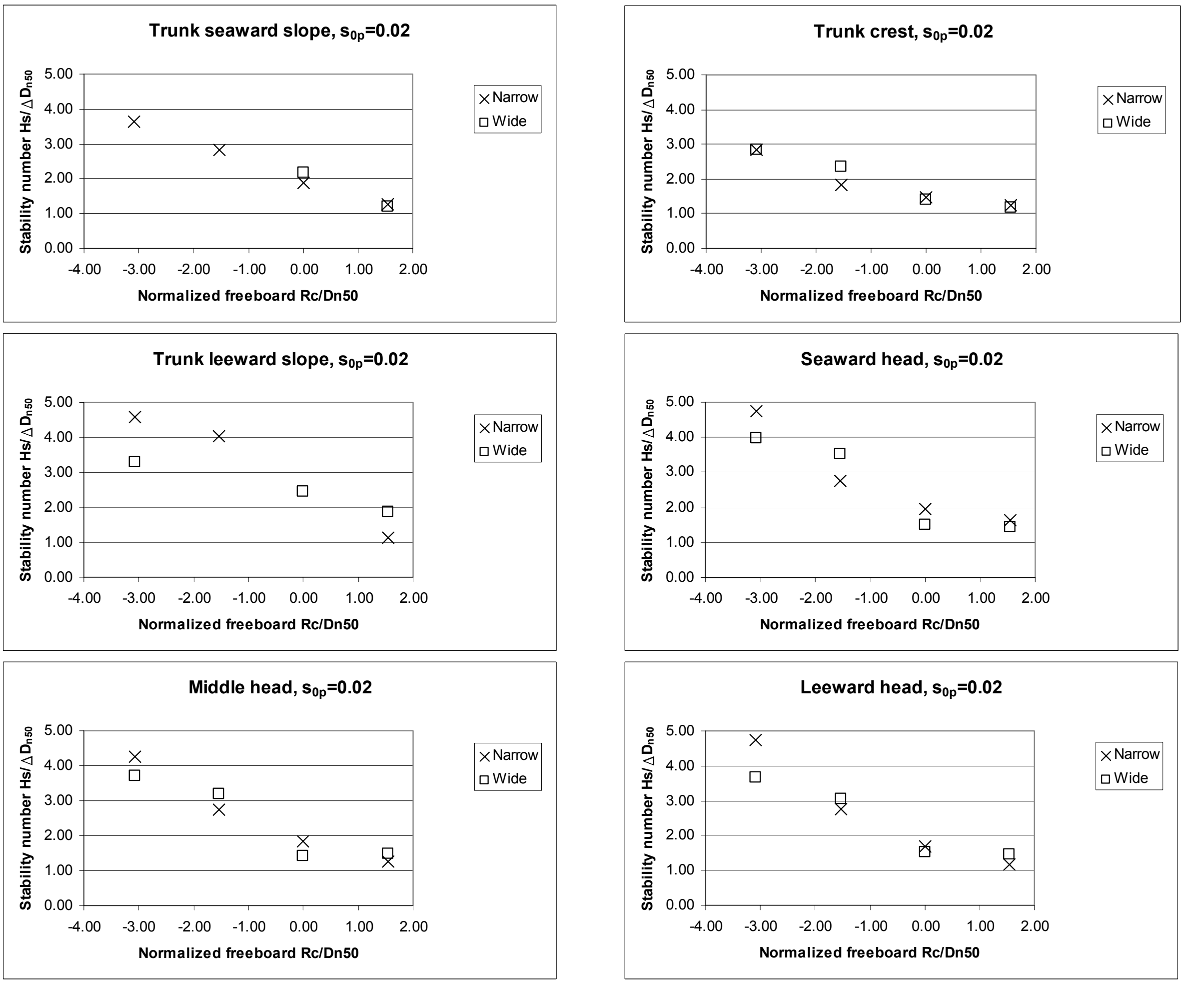


Figure D.19. Stability related to crest width. Initiation of damage.

D.7.3 Stability related to obliquity

The wide structure was tested in waves with oblique main directions. Obliquities -20°,-10°, +10°, and +20° are compared to normal incidence waves (0°) in Figure D.20.

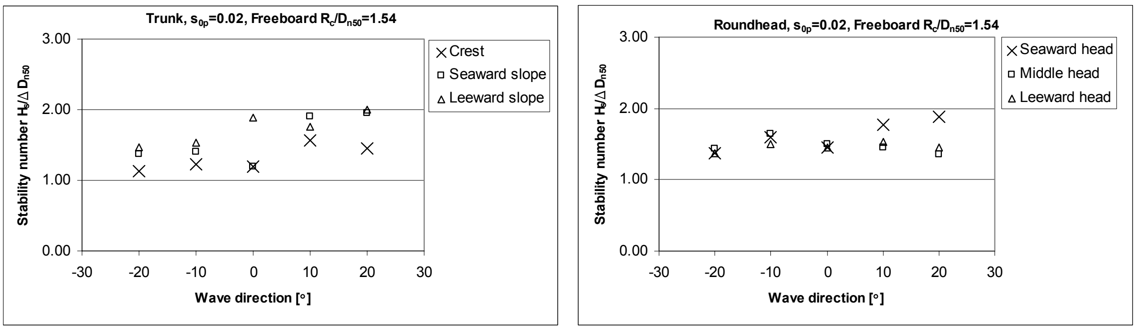


Figure D.20. Stability related to wave direction. Initiation of damage. Left: Trunk. Right: Roundhead.

The trunk is the least stable under normal incidence waves. The crest is the least stable part of the trunk under all wave directions. Figure D.20 (left) is not completely symmetric. The reason is that the layout in the basin is not symmetric. When waves with main direction < 0° are generated, waves are getting trapped between structure, side walls and paddles. This causes a less accurate description of the incoming waves.

When waves with main direction > 0° are generated the seaward head is becoming significantly more stable (see Figure D.20, right). The stability of the leeward and middle head is only slightly altered, but the middle head is becoming slightly less stable than the leeward head. During the experiments it was experienced (as described in chapter D.4) that wave breaking tend to focus at the roundhead forming a jet of water and air slamming down on the top part of leeward head. This effect shifted towards the middle head in case of oblique waves causing the middle head more prone to damage.

D.8 Experimental data compared to existing formulae

In the following the test results from the AAU tests are compared to the formulae by Powell and Allsop (1985), Van der Meer (1990), and Vidal *et al*. (1992, 1995, 2000). Formulae and explanations are given in Appendix F.

D.8.1 Powell and Allsop (1985), trunk front slope

Only two test series in the AAU tests can be compared to the formula by Powell and Allsop. In the AAU experiments the largest freeboard to water depth ratio was $R_c/h = 0.2$ (freeboard +0.05 m, water depth 0.25 m). In tests 1-4 (narrow structure) and 37-40 (wide structure) normal incidence waves with $s_{0p} = 0.02$ were generated.

The first row in Table F.1 (Appendix F) with $R_c/h = 0.29$ corresponds to a slightly more emerged structure than $R_c/h = 0.2$ in the AAU test. However values for $R_c/h = 0.29$ are used for comparison. Therefore the stability according to the Powell and Allsop formula in Eq (F.1) should be the same or slightly smaller than for the front slope in the AAU experiments. The parameters *a* and *b* in the first row of Table F.1 are used together with $s_{0p} = 0.02$, and the curve on Figure D.21 is established. It is seen that the Powell and Allsop formula follows the experimental data up to $N_{od}/N_a \cong 0.03$ (corresponding to 3% displaced armour units). For higher damage levels the formula predicts lower stability than what was measured.

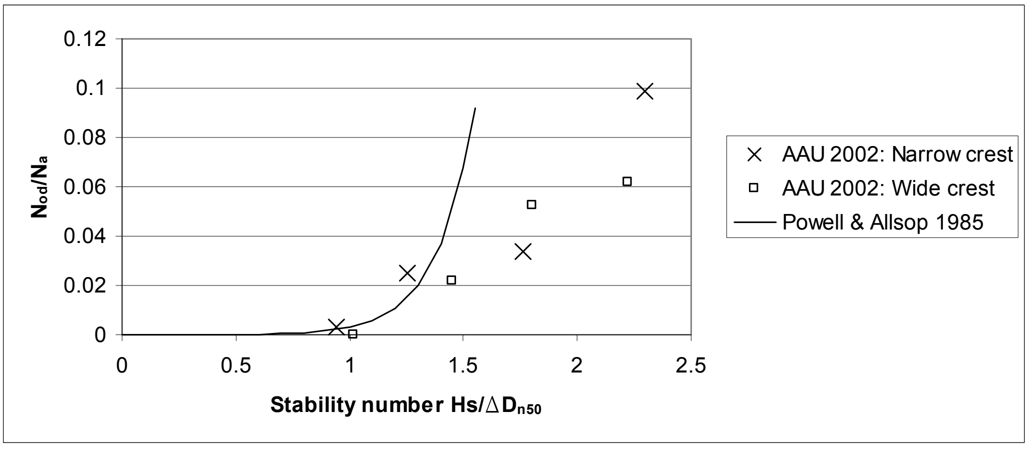


Figure D.21. Test results compared to formulae by Powell & Allsop (1985).

At initiation of damage $N_{od}/N_a = 0.01$ (1% displaced stones) the Powell & Allsop formula gives $H_s/\Delta D_{n50} = 1.19$. This result is very close to the values obtained in the AAU experiments (1.20 and 1.26).

D.8.2 Van der Meer (1990), trunk front slope, emerged structure

The Van der Meer (1990) formula for low crest slopes given in Eq (F.2) is valid for positive freeboard. The following tests are available for comparison of seaward slope.

Table D.7. Tests to be compared with Van der Meer (1990) formula.

Test	Crest width [m]	s_{0p}	Freeboard [m]
1-4	0.10	0.020	+0.05
5-8	0.10	0.035	+0.05
9-12	0.10	0.020	0.00
13-17	0.10	0.035	0.00
37-40	0.25	0.020	+0.05
60-63	0.25	0.020	0.00

In the Van der Meer (1990) formula the Broderick (1983) damage parameter S is used to quantify the damage. For the AAU tests the relationship between damage parameter S and number of displaced units is $S = 0.11N$ (see Chapter 0). The S-values corresponding to the number of displaced units in the AAU tests are calculated from this equation.

Table D.8. Parameters used in Van der Meer's 1990 formula for comparison.

Parameter in van der Meer's formula	Value used for comparison	Explanation
P	0.5	For two layer structure
$\tan(\alpha)$	0.5	Structure slope
N_z	1000	Number of waves
$s_{0p} = H_s/L_{0p}$	0.02/0.035	Wave steepness' s_{0p} = 0.02 or s_{0p} = 0.035 are used depending on experiment. The actual s_{0p}'s varies a little in the tested wave conditions but are close to these values.
$s_m = H_s/L_{0m}$	$1.5 \cdot s_{0p}$ (0.03/0.05)	Wave steepness' s_m = 0.03 or s_m = 0.05 are used depending on experiment. The actual s_m's varies a little in the tested wave conditions but are close to these values.

Van der Meer suggests replacing H_s by $H_{2\%}/1.4$ in case of depth-limited waves. The actual significant wave heights in the experiments are close to this value ($H_s \cong H_{2\%}/1.3$, see Figure D.13) and no replacement has therefore been performed.

The reduction factors f_i are first calculated from Eq (F.2). The reduction factors are then used in Eq (F.4) to calculate the damage S, see Figure D.22 and Figure D.23. All tests were in the plunging wave regime, i.e. $\xi_m < \xi_{mc}$.

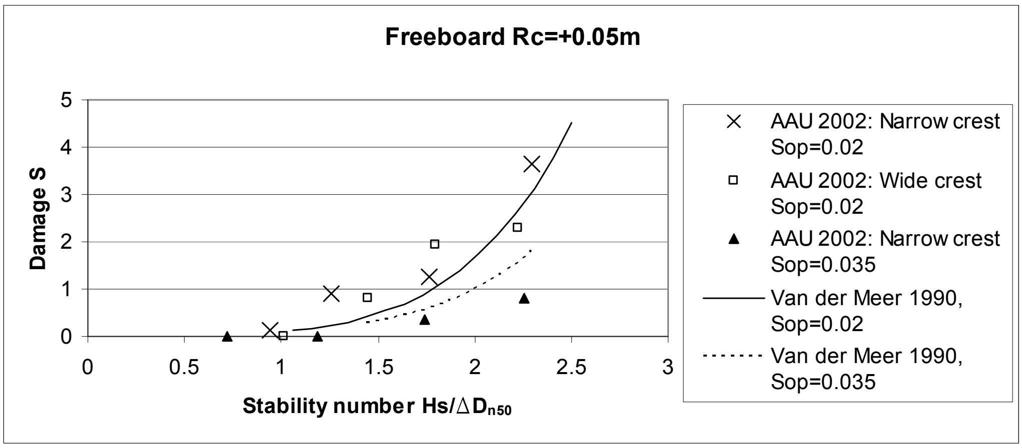


Figure D.22. Tests for front slope compared to Van der Meer (1990) formula, positive freeboard.

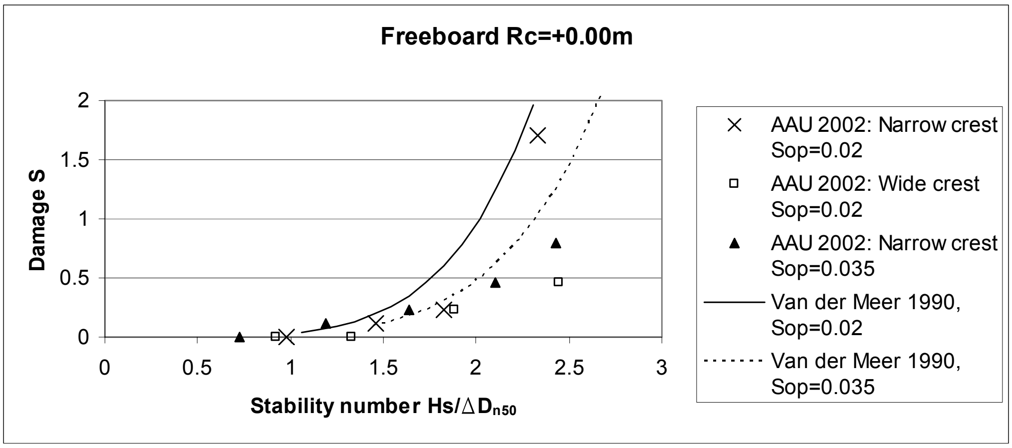


Figure D.23. Tests for front slope compared to Van der Meer (1990) formula, zero freeboard.

The Van der Meer 1990 formula for low crest slopes gives approximately the same stability numbers for initiation of damage as measured in the experiments, and the curves follows the trend of the data.

D.8.3 Van der Meer (1990), trunk, submerged structure

The Van der Meer (1990) formula given in Appendix F, Eq (F.6), is used for comparison with the tests with zero or negative freeboard. The procedure is the same as used in chapter D.8.2. The following tests are available for comparison.

Table D.9. Tests to be compared with Van der Meer 1990 formula.

Test	Crest width [m]	s_{0p}	Freeboard [m]
9-12	0.10	0.020	0.00
13-17	0.10	0.035	0.00
18-22	0.10	0.020	-0.05
23-27	0.10	0.035	-0.05
28-31	0.10	0.020	-0.10
32-36	0.10	0.035	-0.10
60-63	0.25	0.020	0.00
64-66	0.25	0.020	-0.05
67-69	0.25	0.020	-0.10

Damage S in the AAU tests is calculated from the modified Broderick equation $S = 0.11N$ (see Chapter 0), where the number of displaced stones N is found as the sum of the number of displaced stones on the seaward slope, the crest and the leeward slope.

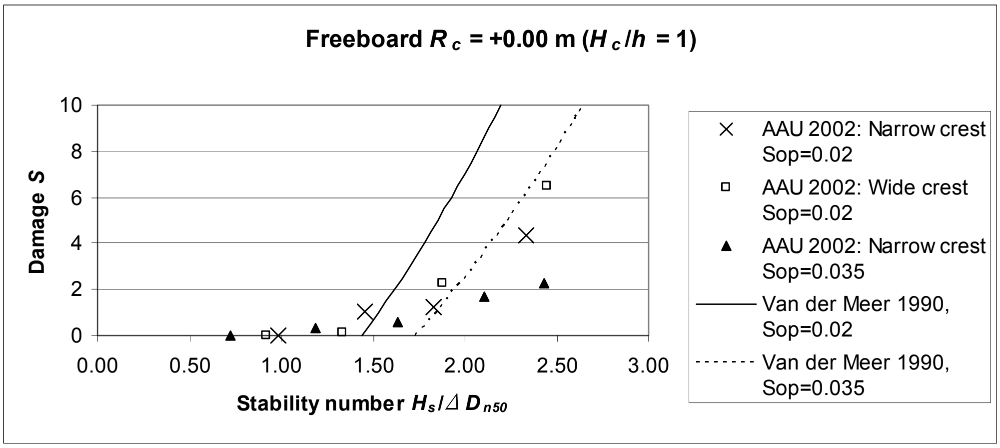


Figure D.24. Tests for trunk compared to Van der Meer (1990) formula, zero freeboard.

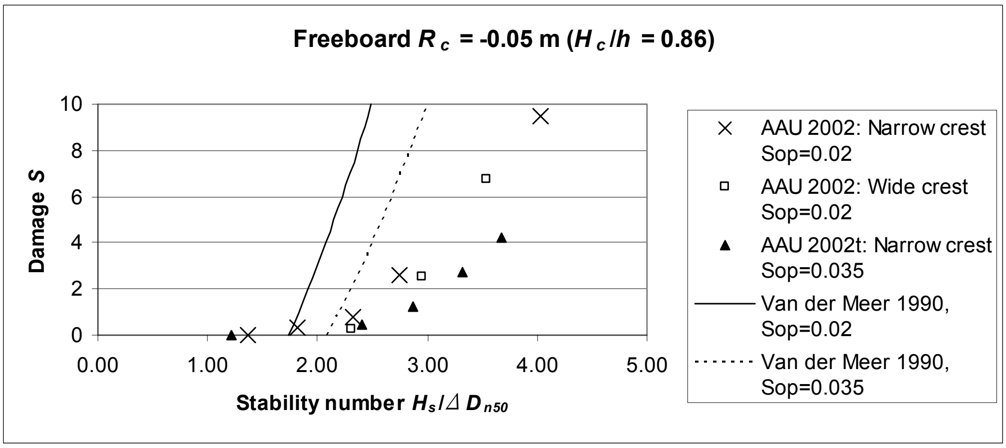


Figure D.25. Tests for trunk compared to Van der Meer (1990) formula, freeboard R_c = -0.05 m.

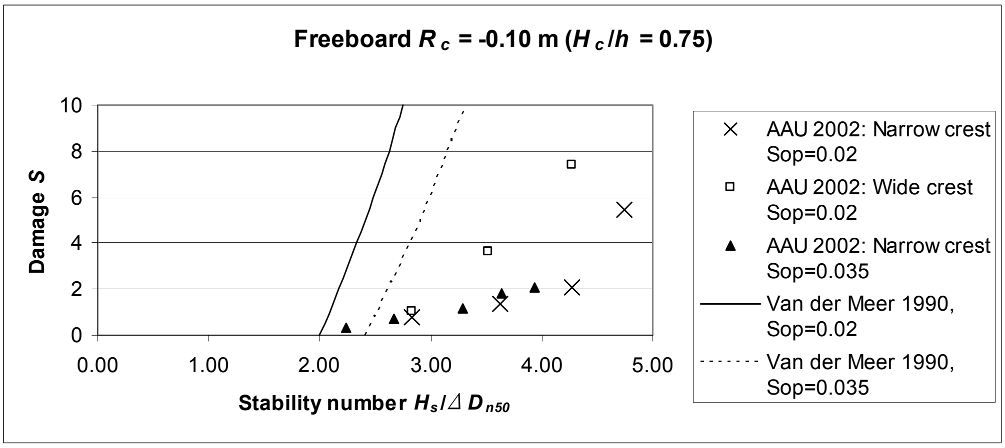


Figure D.26. Tests for trunk compared to Van der Meer (1990) formula, freeboard R_c = -0.10 m.

The formula does not fit the data well especially for the most submerged structure. In general the curves are too steep predicting too much damage. For zero freeboard (Figure D.24) the predicted stability number for start of damage (S = 0 to 2) is close to the test results but for larger submergence (Figure D.25 and Figure D.26) the predicted stability number for start of damage is lower than found in the AAU test results.

D.8.4 Vidal *et al.* (1992, 1995, 2000), head and trunk stability

Vidal *et al.* (2000) proposed parameterized curves corresponding to initiation of damage of the trunk and the head of low-crested and submerged breakwaters. The formula is given in Appendix E, Eq (F.7). Vidal *et al.* (1995) defined the damage S corresponding to initiation of damage in Eq (F.7) as $S = 1$ for the trunk crest and the seaward slope, and $S = 0.5$ for the trunk leeward slope. Vidal divided the roundhead in two sections; the front head and the back head. The front head covered 60° of the seaward part of the roundhead (corresponding to the seaward head in the AAU tests) and the back head covered the remaining 120° (corresponding to the combined middle and leeward head in the AAU tests). A methodology to calculate damage S_{head} for the roundhead sections was proposed by Vidal *et al.* (1995) as described in Chapter 3.4, in which initiation of damage for the head was defined as $S = 1$ for the back head and the front head.

In the present tests initiation of damage was defined as the damage level where 1% of the stones in a section are displaced. This degree of damage corresponds to a lower damage level than the level used by Vidal *et al.* However, the formula proposed by Vidal *et al.* (2000) is compared directly to the test results in the following figures without any corrections of the damage level.

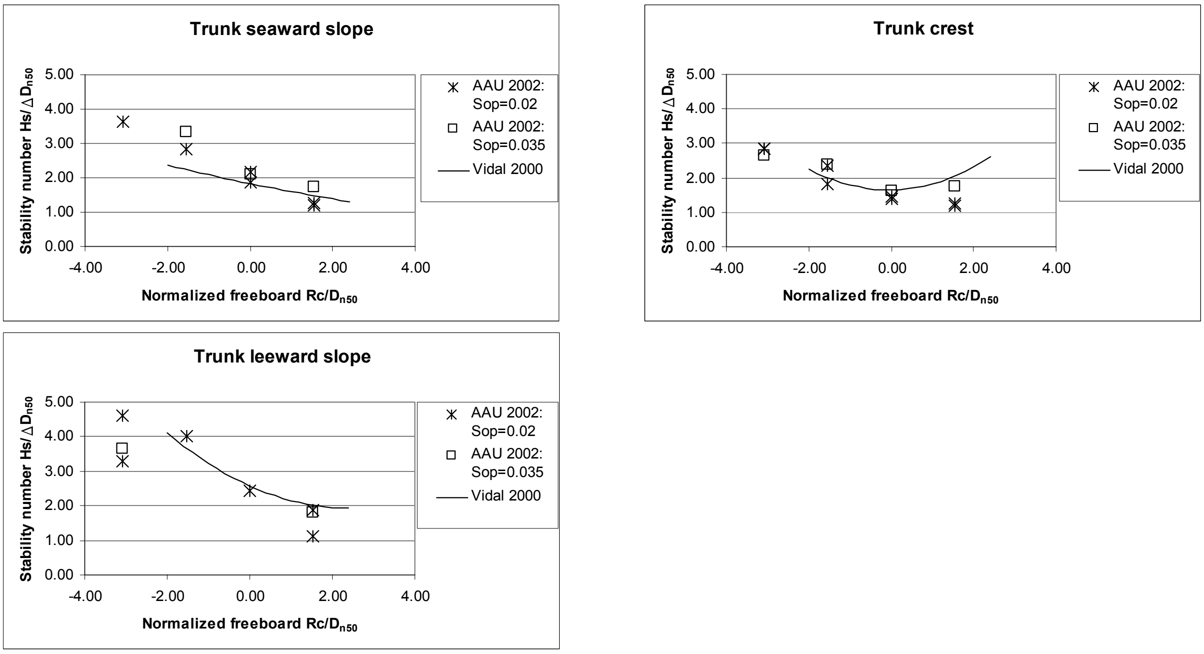


Figure D.27. Tests for trunk compared to the Vidal *et al.* (2000) parameterized formula.

It is seen that the Vidal *et al.* (2000) formula for the trunk fits the data quite well. However, the trunk seaward slope under submerged conditions tends to be a bit more stable in the AAU tests.

The test results for both the leeward and the middle head are plotted on Figure D.28 (right).

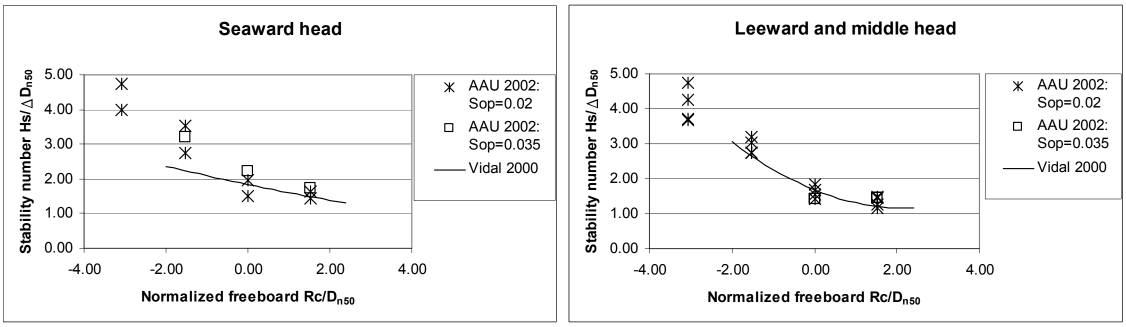


Figure D.28. Tests for roundhead compared to Vidal's parameterized formula.

Vidal's formula for the roundhead fits the data well. However, the seaward head under submerged conditions tends to be a bit more stable in the AAU tests. This will be described in more detail in chapter D.9.3, in which Vidal's test data are compared to the AAU tests with the same definition of damage.

D.9 Experimental data compared to existing datasets

In the following the test results from the AAU tests are compared to tests performed at UCA (2001), Delft (1995), NRC (1992), and Delft (1988). Details about all tests are given in Appendix F.

D.9.1 UCA 2001

In February 2001 stability tests were carried out in the wave flume at University of Cantabria. Details about the tests are given in Appendix F. A homogeneous cross section with crest height H_c = 0.25 m was tested subject to 16 irregular wave conditions. Water depths h was 0.2 m and 0.3 m corresponding to freeboards -0.05 m and +0.05 m. The crest height to water depth ratios H_c/h were approximately the same as for the AAU tests, see Table D.10.

Table D.10. Differences in crest height for UCA and AAU tests.

	UCA tests			AAU tests for comparison		
	Freeboard R_c [m]	R_c/D_{n50}	H_c/h	Freeboard R_c [m]	R_c/D_{n50}	H_c/h
Submerged crest	-0.05	-4.17	0.83	-0.05	-1.54	0.86
Emerged crest	+0.05	4.17	1.25	+0.05	1.54	1.20

Damage S in the AAU tests have been calculated from the modified Broderick equation $S = 0.11N$ (see Chapter 0), where the number of displaced stones N is found as the sum of displaced stones on the seaward slope, the crest and the leeward slope. The AAU tests given in Table D.11 are available for comparison.

Table D.11. AAU tests available for comparison with UCA tests.

Test	Crest width [m]	s_{0p}	Freeboard [m]
1-4	0.10	0.020	+0.05
5-8	0.10	0.035	+0.05
37-40	0.25	0.020	+0.05
18-22	0.10	0.020	-0.05
23-27	0.10	0.035	-0.05
64-66	0.25	0.020	-0.05

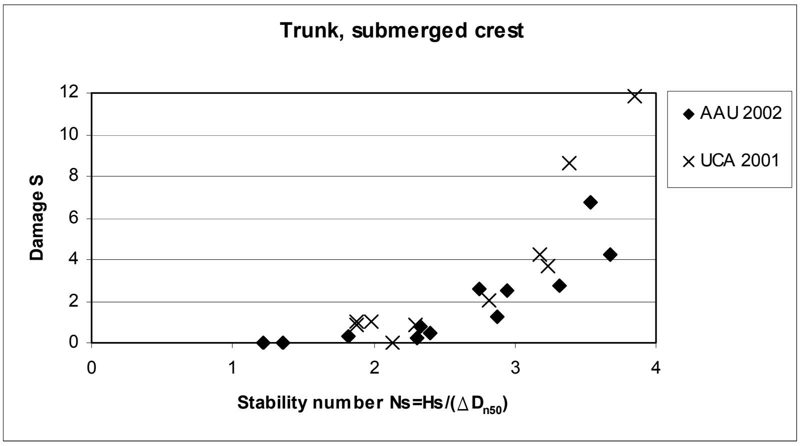


Figure D.29. AAU and UCA tests with freeboard R_c = -0.05 m.

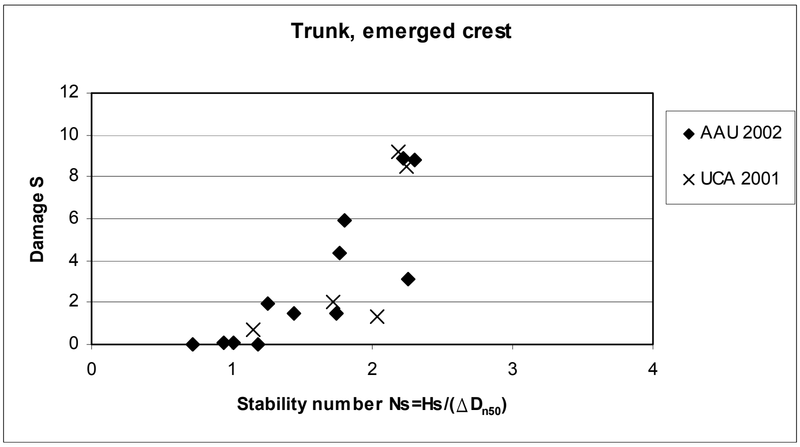


Figure D.30. AAU and UCA tests with freeboard R_c = +0.05 m.

The two data sets are in agreement as the points on Figure D.29 and Figure D.30 follow the same trend. However, very small stones were used in the UCA tests (D_{n50} = 0.012 m), which indicates that viscous scale effects were present in the tests. On the other hand the UCA structure was homogeneous without core. These two deviations from the AAU structures counteracts each other. Caution should therefore be taken when drawing conclusions based on the comparisons.

D.9.2 Delft 1995

Burger (1995) tested the influence of rock shape and grading on the stability of front, crest and rear slope of low-crested structures. Results are presented by Burger (1995) and Van der Meer *et al*. (1996).

Table D.12. Delft 1995 test series to be compared to the AAU tests.

	Delft 1995			AAU tests for comparison		
	Freeboard R_c [m]	R_c/D_{n50}	H_c/h	Freeboard R_c [m]	R_c/D_{n50}	H_c/h
Emerged crest	+0.05m	2.0	1.12	+0.05m	1.54	1.2

The Delft 1995 results are available for two wave steepness's s_{0p} = 0.02 and s_{0p} = 0.04, which is approximately the same as used in the AAU tests. Damage S in the AAU tests has been calculated from the modified Broderick equation $S = 0.11N$ (see Chapter 0), where the number of displaced stones N is found as the sum of displaced stones on the seaward slope, the crest and the leeward slope. The AAU tests given in Table D.13 are available for comparison.

Table D.13. AAU tests available for comparison with Delft 1995 tests.

Test	Crest width [m]	s_{0p}	Freeboard [m]
1-4	0.10	0.020	+0.05
5-8	0.10	0.035	+0.05
37-40	0.25	0.020	+0.05

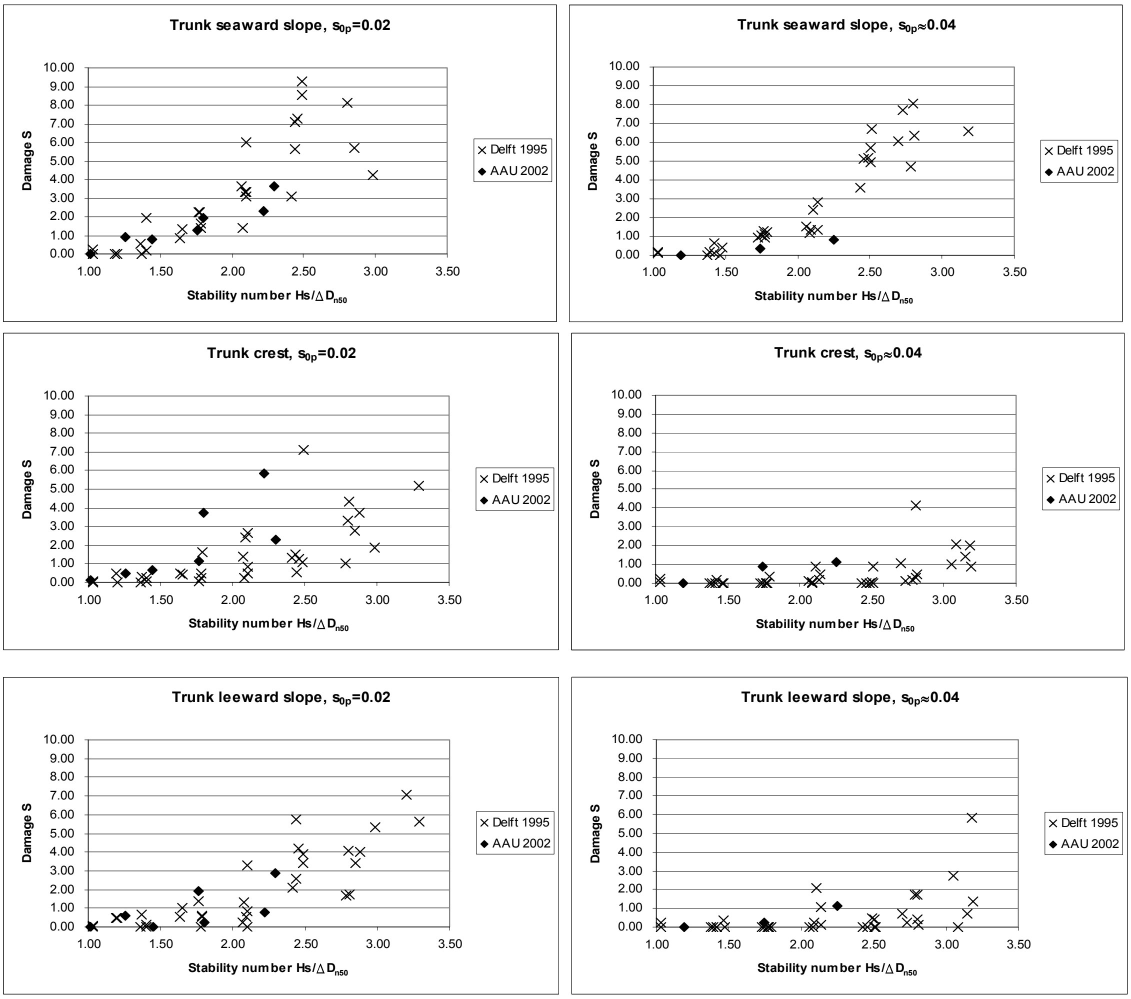


Figure D.31. Stability of trunk sections, Delft 1995 tests compared to AAU tests.

In Figure D.31 it is seen that the two datasets are in agreement with respect to trends. However, the crest seems to be slightly more prone to damage in the AAU tests, and the seaward slope seems to be slightly less prone to damage in the AAU tests. This could be due to different definition of the areas covered by the trunk sections. In the AAU tests the definition of sections in Figure D.7 (on page 140) was adopted, whereas in the Delft 1995 tests the seaward and the leeward slopes were extended to the surface of the crest. To investigate whether this could influence the results the total damage for the trunk was calculated as the sum of damage to the seaward slope, crest, and leeward slope, see Figure D.32.

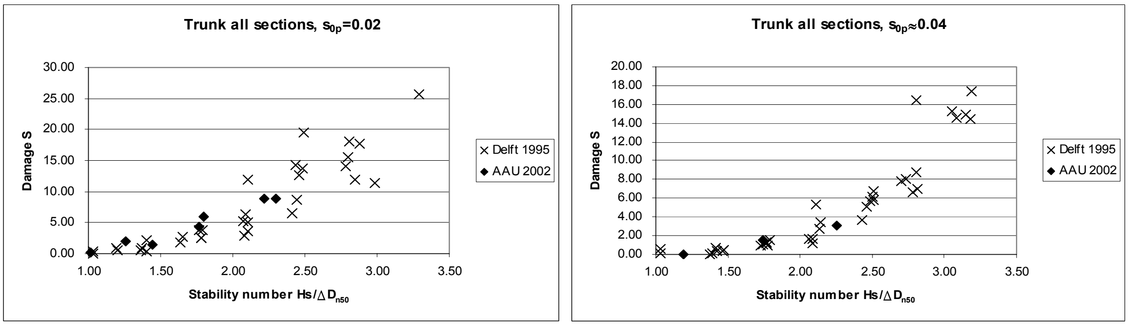


Figure D.32. Stability of total trunk section, Delft 1995 tests compared to AAU tests.

In Figure D.32 it is seen that the two datasets are in almost perfect agreement. The differences in Figure D.31 are therefore believed to be due to the different definitions of sections.

D.9.3 NRC 1992

Vidal *et al.* performed 3D stability tests at the Hydraulics Laboratory of the National Research Council Canada (NRC) in Ottawa, Canada, 1991-1992 on a complete 4.7 m long structure in irregular head on waves. Detailed description of setup is found in Vidal *et al.* (1995) and in Appendix F.

Structure heights and freeboards were different in the AAU tests and the NRC tests. However the following three test series are compared in the following. In the first test series in Table D.14 R_c/D_{n50} = -2.0 (NRC tests) and R_c/D_{n50} = -1.54 (AAU tests). This means that the AAU tests on submerged structure will be compared to a relatively more submerged structure in the NRC tests. According to this less damage for the same stability number is expected for the submerged NRC structure.

Table D.14. NRC test series to be compared to the AAU tests.

	NRC tests			AAU tests for comparison		
	Freeboard R_c [m]	R_c/D_{n50}	H_c/h	Freeboard R_c [m]	R_c/D_{n50}	H_c/h
Submerged crest	-0.05	-2.0	0.89 / 0.92	-0.05	-1.54	0.86
Zero freeboard	0	0	1	0	0	1
Emerged crest	+0.04	1.61	1.07	+0.05	1.54	1.2

Table D.15. AAU tests available for comparison with NRC tests.

Test	Crest width [m]	s_{0p}	Freeboard [m]
1-8	0.10	0.02/0.035	+0.05
9-17	0.10	0.02/0.035	0
18-27	0.10	0.02/0.035	-0.05
37-40	0.25	0.02	+0.05
60-63	0.25	0.02	0
64-66	0.25	0.02	-0.05

For the trunk it is chosen only to compare results for the crest stability.

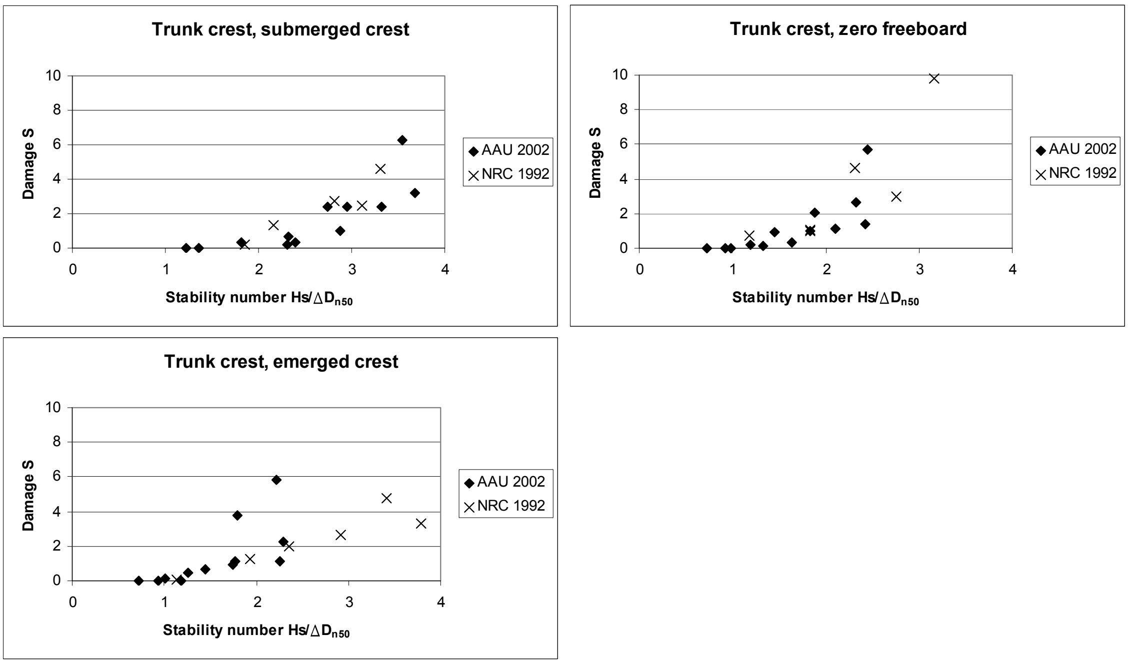


Figure D.33. Trunk crest stability, NRC tests compared to AAU tests.

In Figure D.33 it is seen that the points for the two data sets follow the same trend. However, the crest under submerged conditions does not seem to be less prone to damage for the NRC tests. This indicates that the crest seems to be slightly more stable in the AAU tests under submerged conditions.

To compare damage to the roundhead the numbers of displaced stones in the AAU sections have been converted to the damage *S*-value as used in the NRC tests. The methodology described in Chapter 3.4 has been used. On the following figures the leeward head corresponds to the same section for the leeward head as used in the NRC tests. The leeward head on the following figures therefore corresponds to the combined area of the middle and leeward head in the AAU tests. Please note the different scaling of the axes on the following figures.

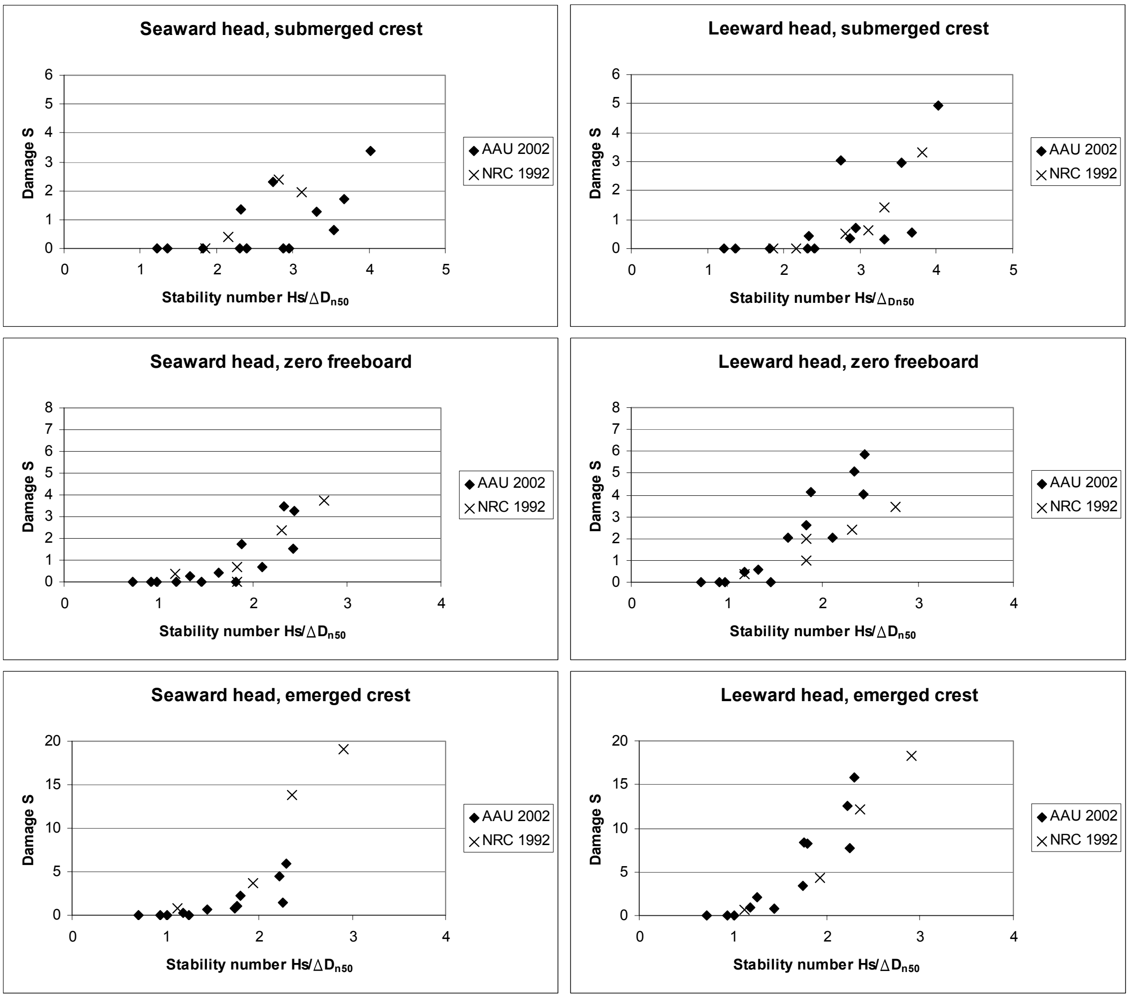


Figure D.34. Roundhead stability, NRC tests compared to AAU tests.

The data points on Figure D.34 are in agreement. However, as for the trunk crest, the NRC structure does not seem to be less prone to damage under submerged conditions. There can be several explanations for that. The main differences to the present tests are (in subjectively estimated order of priority) described in the following and the influence of the parameters that are believed to be of most importance is explained further.

- The structure slopes were 1:1.5 in the NRC tests (1:2 in AAU tests)
  - A steeper slope is less stable. This indicates that the NRC structure should be less stable than the AAU structure.
- No foreshore slope was present in NRC tests
  - In the NRC tests a horizontal seabed was used. On a horizontal seabed it is not possible to produce as steep waves as on a sloping foreshore. This can make the structure on the horizontal seabed more stable. Therefore the stability of the NRC structure could be larger than the AAU structure.
- 2D irregular waves were generated in NRC tests (3D in AAU tests)
- Higher structure in NRC tests (40-60 cm in NRC tests and 30 cm in AAU tests)
- Slightly smaller stones in NRC tests (D_{n50} = 2.5 cm in NRC tests and D_{n50} = 3.3 cm in AAU tests)

D.9.4 Delft 1988

Van der Meer (1988) performed LCS stability tests in the wave flume at Delft Hydraulics. Water depth was kept constant and structure height was changed.

Table D.16. Delft 1988 test series to be compared to the AAU tests.

	Delft 1988			AAU tests for comparison		
	Freeboard R_c [m]	R_c/D_{n50}	H_c/h	Freeboard R_c [m]	R_c/D_{n50}	H_c/h
Submerged crest	-0.10	-2.91	0.75	-0.10	-3.08	0.75
Zero freeboard	0	0	1	0	0	1
Emerged crest	+0.125	3.63	1.31	+0.05	1.54	1.2

Damage S in the AAU tests has been calculated from the modified Broderick equation $S = 0.11N$ (see Chapter 0), where the number of displaced stones N is found as the sum of displaced stones on the seaward slope, the crest and the leeward slope. The AAU tests given in Table D.17 are available for comparison.

Table D.17. AAU tests available for comparison with Delft 1988 tests.

Test	Crest width [m]	s_{0p}	Freeboard [m]
1-8	0.10	0.02/0.035	+0.05
9-17	0.10	0.02/0.035	0
28-36	0.10	0.02/0.035	-0.10
37-40	0.25	0.02	+0.05
60-63	0.25	0.02	0
67-69	0.25	0.02	-0.10

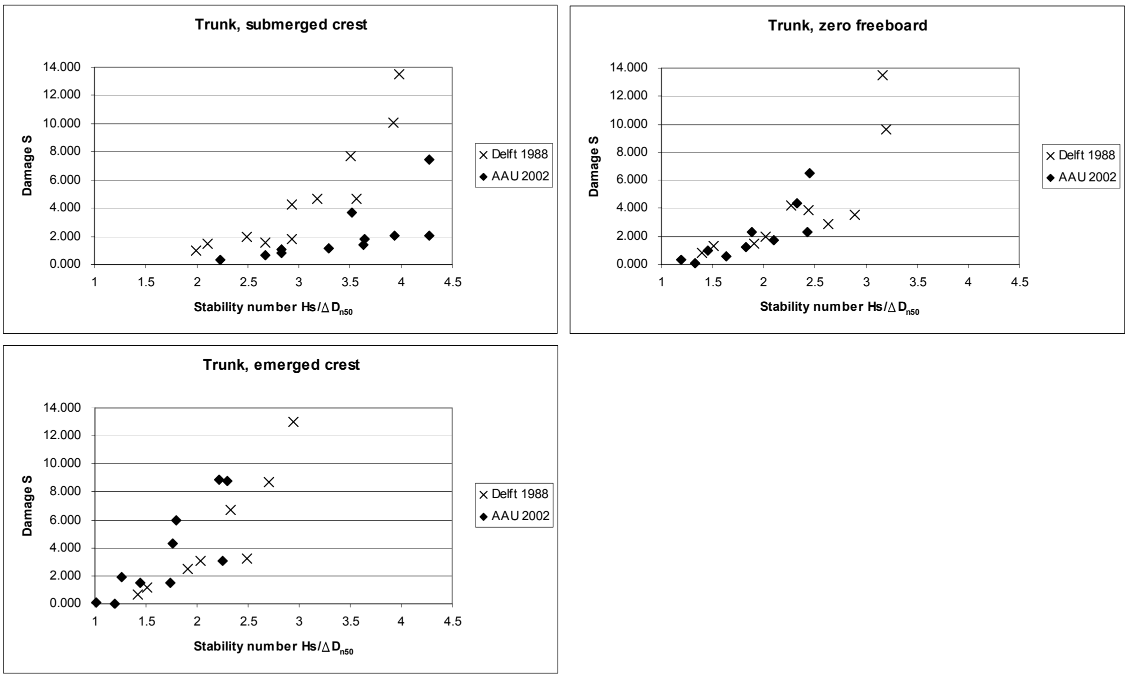


Figure D.35. Trunk stability, Delft 1988 tests compared to AAU tests.

The two datasets are in agreement for zero freeboard and emerged crest. However, under submerged conditions the Delft 1988 structure was more prone to damage.

In Table D.16 it is seen that for submerged crest the relative submergence R_c/D_{n50} = -3.08 in the AAU test and R_c/D_{n50} = -2.91 as target in the Delft 1988 test. In the Delft 1988 tests the actual crest height as built was slightly different from the target. For the submerged crest the actual crest height as built was measured to 0.31 m (taken as the average for the tests with submerged crest). Hereby the actual relative submergence in the Delft 1988 tests was R_c/D_{n50} = -2.62, which is somewhat different from the compared AAU tests.

When the difference in relative freeboard is considered the two datasets are in agreement.

D.10 Conclusions on comparisons of test data and formulae

The AAU test results have been compared to four different test series performed by other researchers. Structure geometries, wave basin/flume layouts, stone characteristics and types of waves generated were different in all four datasets. Because of this some deviations between the results is expected and also observed. However, when the differences are kept in mind all four datasets are considered to be in reasonable agreement with the AAU tests.

The AAU test results were compared to the formula shown in Table D.18. Even though there are differences between tests and formulae, the existing formulae are able to predict the damage in the AAU tests to some extend. As very few tests have been available for comparisons Table D.18 should not be used to check the validity of a certain formula.

Table D.18. Overview of stability formula compared to AAU tests.

Author	Formula valid for	How well does the formula fit the AAU test?	
		For start of damage	For progress of damage
Powell and Allsop (1985)	Trunk front slope	Well	Formula overestimates the progress of damage
Van der Meer (1990)	Trunk front slope, emerged structure	Well	Well
Van der Meer (1990)	Trunk, submerged structure	Formula underestimates the stability in case of large submergence	Formula overestimates the progress of damage
Vidal *et al.* (1992, 1995, 2000)	Head and trunk stability	Well	Well

D.11 Conclusions on the importance of the investigated parameters

The following main conclusions about the AAU tests related to initiation of damage can be drawn:

Influence of	Importance	Comments
Freeboard	Large	A submerged structure is significantly more stable than an emerged low crested structure. The more submerged the more stable. For larger emergence than tested in the AAU tests the overtopping will reduce and consequently the trunk leeward slope and crest will become more stable.
Crest width	Small	The stability of much wider structures than the tested ones might be larger compared to the tested relatively narrow structures.
Wave steepness	Small	Long waves (s_{0p} = 0.020) cause only slightly larger damage to the structure than steeper waves (s_{0p} = 0.035) for low damage levels. However for higher damage levels the structure becomes relatively more stable in steep waves.
Obliquity of waves	Small	All parts of the trunk are slightly more stable under oblique wave attack than under normal incidence wave attack. The stability of the roundhead sections in case of oblique waves < 0° (a large part of the head exposed to direct wave attack) is the same as for normal incidence waves. The stability of the leeward and middle part of the roundhead in case of oblique waves > 0° (when a large part of the head is in lee of direct wave attack) is the same as for normal incidence waves, but the area of damage shifts towards the middle part of the head. However the seaward part of the head is becoming significantly more stable.

The conclusions can only be applied within the tested range of parameters.

Appendix E Tabulated data of 3D tests at AAU

More detailed information can be found in the databank for the tests on the CD. The following parameters are used in the tables:

β target incoming wave direction (0° is normal incidence waves)

B crest width

R_c freeboard

wave steep., H_s and T_p target deepwater wave characteristics

h water depth

Hs(3) significant wave height from gauge 3 in the 3-wavegauge system

H_{moI} incident significant wave height from the array

No waves is the average number of waves from the 3-wavegauge system

Spr. and β *(Hi)* spreading and direction of the incoming waves from the array, respectively.

N number of displaced stones counted from the photos. Colour coding:

R: Red, **G**: Green, **B**: Blue, **Y**: Yellow, **W**: White, **K**: Black

Table E.1. Details about 3D stability tests at Aalborg University. Narrow cross-section.

Target								Measured waves					Damage, trunk			Damage, roundhead		
Test no.	β [°]	B [m]	Rc [m]	Wave steep.	Hs [m]	Tp [s]	h at LCS [m]	Hs (3) [m]	Hmo_I [m]	No waves	Spr. [°]	β (Hi) [°]	SS N	C N	LS N	SH N	MH N	LH N
1	0	0.1	0.05	0.02	0.05	1.27	0.25	0.049	0.051	1034	10	-3	1G	0	0	0	0	0
2	0	0.1	0.05	0.02	0.075	1.55	0.25	0.065	0.076	1082	12	-1	8Y	4G	5B	0	2G+1K	3B+1G
3	0	0.1	0.05	0.02	0.1	1.79	0.25	0.092	0.103	1052	13	-1	9Y+2B	5R+5G	16B+1Y	3R+1Y	11G+3K	7B+9Y
4	0	0.1	0.05	0.02	0.125	2.00	0.25	0.120	0.117	1129	15	-2	2W+7B+23Y	10R+10G	24B+1Y	10Y+14R	19G+9K+2W	18B+16Y
5	0	0.1	0.05	0.04	0.05	0.90	0.25	0.037	0.037	1055	9	-1	0	0	0	0	0	0
6	0	0.1	0.05	0.04	0.075	1.10	0.25	0.062	0.062	981	9	-1	0	0	0	1R	2G	1B
7	0	0.1	0.05	0.04	0.1	1.27	0.25	0.091	0.084	987	11	1	1B+2Y	5R+3G	2B	2R+1Y	4G+3K	4B+1Y
8	0	0.1	0.05	0.04	0.125	1.42	0.25	0.117	0.110	971	13	-1	4B+3Y	6R+4G	10B	5R+1Y	4G+4K	13B+10Y
9	0	0.1	0	0.02	0.05	1.27	0.3	0.051	0.055	970	9	0	0	0	0	0	0	0
10	0	0.1	0	0.02	0.075	1.55	0.3	0.076	0.080	1003	10	1	1Y	6R+2G	0	0	0	0
11	0	0.1	0	0.02	0.1	1.79	0.3	0.095	0.109	994	11	2	1B+1Y	7R+2G	0	0	2G	4B+1Y
12	0	0.1	0	0.02	0.125	2.00	0.3	0.121	0.130	1044	14	2	1W+2B+12Y	16R+7G	0	11R	6G	6B+4Y
13	0	0.1	0	0.04	0.05	0.90	0.3	0.038	0.038	991	9	0	0	0	0	0	0	0
14	0	0.1	0	0.04	0.075	1.10	0.3	0.062	0.067	991	9	-1	1Y	2R	0	0	0	1B
15	0	0.1	0	0.04	0.1	1.27	0.3	0.085	0.088	978	10	1	2Y	2R+1G	0	1R	0	4B+1Y
16	0	0.1	0	0.04	0.125	1.42	0.3	0.109	0.112	986	11	3	1B+3Y	6R+4G	1B	2R	1G	4B+1Y
17	0	0.1	0	0.04	0.15	1.55	0.3	0.126	0.135	1016	12	0	1B+6Y	7R+5G	1B	5R	3G	6B+4Y
18	0	0.1	-0.05	0.02	0.075	1.55	0.35	0.071	0.083	1004	10	-1	0	0	0	0	0	0
19	0	0.1	-0.05	0.02	0.1	1.79	0.35	0.095	0.111	1004	10	0	0	3R	0	0	0	0
20	0	0.1	-0.05	0.02	0.125	2.00	0.35	0.121	0.138	1058	13	1	1Y	4R+2G	0	3R	1G	0
21	0	0.1	-0.05	0.02	0.15	2.19	0.35	0.143	0.157	1055	13	-1	2Y	13R+8G	0	6R	1W+3G+1K	3B
22	0	0.1	-0.05	0.02	0.175	2.37	0.35	0.209	0.188	1053	14	0	1W+4B+13Y	31R+31G	2Y+1B	2Y+11R	1W+1K+8G	6B+3Y
23	0	0.1	-0.05	0.04	0.1	1.27	0.35	0.063	0.105	999	11	-1	0	0	0	0	0	0
24	0	0.1	-0.05	0.04	0.125	1.42	0.35	0.125	0.134	1008	10	1	1W	1R+2G	0	0	0	0
25	0	0.1	-0.05	0.04	0.15	1.55	0.35	0.149	0.163	1033	11	2	1W	6R+3G	1Y	0	1G	0
26	0	0.1	-0.05	0.04	0.175	1.67	0.35	0.173	0.179	1055	11	-1	1W+1B+2Y	10R+11G	1Y	4R	1G	0
27	0	0.1	-0.05	0.04	0.2	1.79	0.35	0.191	0.193	1005	13	-1	1W+2B+5Y	13R+15G	1Y	1W+1Y+4R	2G	0
28	0	0.1	-0.1	0.02	0.125	2.00	0.4	0.147	0.157	1060	12	2	1Y	5R+1G	0	1R	2G	0
29	0	0.1	-0.1	0.02	0.15	2.19	0.4	0.189	0.185	1025	14	1	3Y	8R+1G	0	1R	2G	1Y
30	0	0.1	-0.1	0.02	0.175	2.37	0.4	0.222	0.203	1057	15	1	5Y	11R+1G	1B	1R	4G+1K	1B+1Y
31	0	0.1	-0.1	0.02	0.2	2.53	0.4	0.247	0.210	1000	17	2	5Y	25R+14G	2B+1Y+1W	3R	4K+5G	3B
32	0	0.1	-0.1	0.04	0.125	1.42	0.4	0.116	0.126	1012	10	1	0	2R	1B	0	1G	0
33	0	0.1	-0.1	0.04	0.15	1.55	0.4	0.139	0.155	1028	10	2	1Y	2R+1G	2B	0	1G	0
34	0	0.1	-0.1	0.04	0.175	1.67	0.4	0.171	0.182	1058	11	3	1Y	5R+1G	2B+1Y	0	1G	0
35	0	0.1	-0.1	0.04	0.2	1.79	0.4	0.189	0.203	1049	12	3	1Y	7R+4G	2B+2Y	0	1G	0
36	0	0.1	-0.1	0.04	0.225	1.90	0.4	0.204	0.201	1027	13	2	1Y	9R+4G	2B+2Y	0	3G+1K	0

Table E.2. Details about 3D stability tests at Aalborg University. Wide cross-section.

Target								Measured waves					Damage, trunk			Damage, roundhead		
Test	β	B	Rc	Wave	Hs	Tp	h at	Hs (3)	Hmo_I	No	Spr.	β (Hi)	SS	C	LS	SH	MH	LH
no.	[°]	[m]	[m]	steep.	[m]	[s]	LCS [m]	[m]	[m]	waves	[°]	[°]	N	N	N	N	N	N
37	0	0.25	0.05	0.02	0.05	1.27	0.25	0.053	0.048	990	10	-1	0	1R	0	0	0	0
38	0	0.25	0.05	0.02	0.075	1.55	0.25	0.075	0.072	1043	12	1	1W+3B+3Y	6G	0	1R+2Y	0	4Y
39	0	0.25	0.05	0.02	0.1	1.79	0.25	0.094	0.096	1029	13	2	1W+5B+11Y	11R+22G	2B	10R+1Y	20G+2K	13B+6Y
40	0	0.25	0.05	0.02	0.125	2.00	0.25	0.116	0.114	1073	16	1	1W+6B+13Y	19R+32G	7B	1W+6Y+17R	6K+26G	21B+14Y
41	-20	0.25	0.05	0.02	0.05	1.27	0.25	0.049	0.046	996	10	-18	0	0	0	0	0	0
42	-20	0.25	0.05	0.02	0.075	1.55	0.25	0.071	0.073	1031	12	-19	1B+2Y	2R+5G	2B	3R	0	3B+2Y
43	-20	0.25	0.05	0.02	0.1	1.79	0.25	0.091	0.094	1039	13	-16	1W+2B+7Y	8R+10G	6B	3Y+9R	16G	16B+5Y
44	-20	0.25	0.05	0.02	0.125	2.00	0.25	0.115	0.108	1100	15	-14	2W+6B+13Y	13R+19G	17B	2W+8Y+22R	6K+22G	22B+6Y
45	-10	0.25	0.05	0.02	0.05	1.27	0.25	0.051	0.047	1021	10	-10	0	0	0	0	0	0
46	-10	0.25	0.05	0.02	0.075	1.55	0.25	0.073	0.074	1027	12	-10	3Y	5G	1B	0	0	0
47	-10	0.25	0.05	0.02	0.1	1.79	0.25	0.097	0.098	1018	14	-8	1B+6Y	3R+16G	8B	1Y+6R	6G	9B+6Y
48	-10	0.25	0.05	0.02	0.125	2.00	0.25	0.119	0.115	1076	16	-8	3B+10Y	4R+18G	13B	1W+2Y+19R	2K+13G	13B+8Y+1W
49	10	0.25	0.05	0.02	0.05	1.27	0.25	0.054	0.049	993	10	8	0	0	0	0	0	0
50	10	0.25	0.05	0.02	0.075	1.55	0.25	0.079	0.072	1017	14	8	0	2G	0	1R	6G	2B
51	10	0.25	0.05	0.02	0.1	1.79	0.25	0.099	0.100	1005	15	11	4Y	5R+6G	5B	4R	3K+15G	13B+8Y
52	20	0.25	0.05	0.02	0.05	1.27	0.25	0.055	0.048	973	11	17	0	0	0	0	0	0
53	20	0.25	0.05	0.02	0.075	1.55	0.25	0.075	0.074	1024	13	19	0	1R+2G	0	1R	7G	2B+1Y
54	20	0.25	0.05	0.02	0.1	1.79	0.25	0.098	0.103	1018	15	21	2Y	3R+4G	2B	1Y+2R	14G	16B+8Y
55	20	0.25	0.05	0.02	0.125	2.00	0.25	0.116	0.117	1077	16	21	3B+4Y	3R+5G	5B	1y+13R	4K+21G	26B+13Y
56	-30	0.25	0.05	0.02	0.05	1.27	0.25	0.050	0.047	980	8	26	0	0	0	0	0	0
57	-30	0.25	0.05	0.02	0.075	1.55	0.25	0.069	0.068	1014	13	23	0	2R	0	0	1K+1G	2B
58	-30	0.25	0.05	0.02	0.1	1.79	0.25	0.092	0.088	1034	12	23	1B+2Y	5R+3G	4B	1R	2K+4G	10B+2Y
59	-30	0.25	0.05	0.02	0.125	2.00	0.25	0.110	0.100	1129	15	20	1B+7Y	9R+17G	17B	1Y+8R	4K+18G	21B+4Y
60	0	0.25	0	0.02	0.05	1.27	0.3	0.048	0.050	1025	9	0	0	0	0	0	0	0
61	0	0.25	0	0.02	0.075	1.55	0.3	0.069	0.074	1023	11	1	0	1R	0	1R	2G	0
62	0	0.25	0	0.02	0.1	1.79	0.3	0.098	0.103	992	13	2	2Y	8R+10G	0	7R	9G	8B
63	0	0.25	0	0.02	0.125	2.00	0.3	0.127	0.124	1023	15	4	4Y	34R+16G	3B	1Y+14R	14G	13B
64	0	0.25	-0.05	0.02	0.1	1.79	0.35	0.120	0.129	985	11	3	0	2R	0	0	0	0
65	0	0.25	-0.05	0.02	0.125	2.00	0.35	0.153	0.157	1075	13	3	0	17R+4G	1B	0	1K	2B
66	0	0.25	-0.05	0.02	0.15	2.19	0.35	0.184	0.179	1031	13	2	2Y	39R+16G	2B	3R	1K+5R	5B+3Y
67	0	0.25	-0.1	0.02	0.125	2.00	0.4	0.147	0.157	1060	12	2	0	5R+3G	1B	0	1G	0
68	0	0.25	-0.1	0.02	0.15	2.19	0.4	0.183	0.180	1026	13	2	0	21R+7G	3B+1Y	0	2G	1Y
69	0	0.25	-0.1	0.02	0.175	2.37	0.4	0.222	0.203	1057	15	1	1B+1Y	37R+19G	4B+1Y	1Y+4R	6G	10B+1G

Table E.3. Stability numbers related to initiation of damage for all 3D tests at Aalborg University.

Test no.	β [°]	Crest width [m]	Free-board [m]	R_c/D_{n50}	Wave steepness	Ns (ID) for trunk SS	C	LS	Ns (ID) for roundhead SH	MH	LH
1-4	0	0.1	0.05	1.54	0.02	1.26	1.26	1.13	1.64	1.26	1.18
5-8	0	0.1	0.05	1.54	0.035	1.74	1.74	1.81	1.74	1.30	1.47
9-12	0	0.1	0	0.00	0.02	1.87	1.46	-	1.97	1.83	1.68
13-17	0	0.1	0	0.00	0.035	2.10	1.64	-	2.21	2.27	1.41
18-22	0	0.1	-0.05	-1.54	0.02	2.83	1.82	4.03	2.75	2.75	2.75
23-27	0	0.1	-0.05	-1.54	0.035	3.32	2.40	-	3.21	-	-
28-31	0	0.1	-0.1	-3.08	0.02	3.63	2.83	4.59	4.75	4.27	4.75
32-36	0	0.1	-0.1	-3.08	0.035	-	2.67	3.64	-	-	-
37-40	0	0.25	0.05	1.54	0.02	1.20	1.19	1.88	1.45	1.50	1.45
41-44	-20	0.25	0.05	1.54	0.02	1.37	1.13	1.47	1.37	1.44	1.37
45-48	-10	0.25	0.05	1.54	0.02	1.40	1.23	1.54	1.60	1.64	1.50
49-51	100	0.25	0.05	1.54	0.02	1.91	1.56	1.75	1.78	1.45	1.54
52-55	110	0.25	0.05	1.54	0.02	1.95	1.45	1.99	1.88	1.36	1.45
56-59	60	0.25	0.05	1.54	0.02	1.77	1.41	1.77	1.85	1.33	1.38
60-63	90	0.25	0	0.00	0.02	2.16	1.40	2.44	1.51	1.41	1.54
64-66	90	0.25	-0.05	-1.54	0.02	-	2.34	-	3.54	3.18	3.05
67-69	90	0.25	-0.1	-3.08	0.02	-	2.83	3.29	3.97	3.71	3.67

Appendix F Existing armour stability formulae

The text given in this appendix has earlier partly been published through the DELOS project, Work Package 2.3, Delivery D22 and D43, see Burcharth and Kramer (2002).

F.1 Powell and Allsop (1985), low crested slopes

Powell and Allsop (1985) analyzed the data by Allsop (1983) and proposed the following stability formula for two-layer armoured overtopped, low-crested slopes:

$$\frac{N_{od}}{N_{ta}} = a \exp\left[b s_p^{-1/3} H_s / (\Delta D_{n50})\right] \quad \text{or} \quad \frac{H_s}{\Delta D_{n50}} = \frac{s_p^{-1/3}}{b} \ln\left(\frac{1}{a}\frac{N_{od}}{N_{ta}}\right) \qquad \text{Eq (F.1)}$$

where values of the empirical coefficients a and b are given in Table F.1 as functions of freeboard R_c and water depth h. N_{od} and N_{ta} are the number of units displaced out of the armour layer and the total number of armour layer units respectively.

Table F.1. Values of coefficients *a* and *b* in Eq (F.1).

R_c/h	$a \cdot 10^4$	b	wave steepness H_s/L_p
0.29	0.07	1.66	< 0.03
0.39	0.18	1.58	< 0.03
0.57	0.09	1.92	< 0.03
0.38	0.59	1.07	> 0.03

F.2 Van der Meer (1990), low crested slopes

Van der Meer (1988, 1990 and 1991) suggested the van der Meer stability formulae for non-overtopped rock slope, Eq (F.4) and Eq (F.5), to be used with D_{n50} replaced by $f_i \cdot D_{n50}$. The reduction factor f_i is in Van der Meer (1990) given as

$$f_i = \left(1.25 - 4.8 \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}}\right)^{-1} \qquad \text{Eq (F.2)}$$

where R_c is the freeboard $s_{op} = H_s/L_{op,}$ and L_{op} is deep water wave length corresponding to the peak wave period. Limits of Eq (F.2) are given by

$$0 < \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}} < 0.052 \qquad \text{Eq (F.3)}$$

Irregular, head-on waves were used to establish the following formulae for **non-overtopped slopes** (van der Meer 1988)

$$\frac{H_s}{\Delta D_{n50}} = 6.2 \cdot S^{0.2} P^{0.18} N_z^{-0.1} \xi_m^{-0.5} \qquad \text{plunging waves: } \xi_m < \xi_{mc} \qquad \text{Eq (F.4)}$$

$$\frac{H_s}{\Delta D_{n50}} = 1.0 \cdot S^{0.2} P^{-0.13} N_z^{-0.1} (\cot\alpha) \xi_m^P \qquad \text{surging waves: } \xi_m > \xi_{mc} \qquad \text{Eq (F.5)}$$

$$\xi_m = s_m^{-0.5} \tan\alpha \qquad \xi_{mc} = \left(6.2 P^{0.31} (\tan\alpha)^{0.5}\right)^{1/(P+0.5)}$$

where H_s significant wave height in front of breakwater
- D_{n50} Equivalent cube length of medium rock
- ρ_s Mass density of rocks
- ρ_w Mass density of water
- Δ Submerged density, $\Delta = (\rho_s/\rho_w) - 1$
- S relative eroded area
- P notional permeability; for three layer conventional breakwater $P = 0.4$, two layer structure $P = 0.5$, and homogeneous structure $P = 0.6$.
- N_z number of waves
- α slope angle
- s_m wave steepness, $s_m = H_s/L_{om}$
- L_{om} deep water wave length corresponding to mean wave period

Validity:

1) Eq (F.4) and Eq (F.5) are valid for non-depth limited waves. For depth-limited waves H_s is replaced by $H_{2\%}/1.4$.
2) For cot $\alpha \geq 4.0$ only Eq (F.4) should be used.
3) $N_z \leq 7{,}500$ after which number equilibrium damage is more or less reached.
4) $0.1 \leq P \leq 0.6$, $0.005 \leq s_m \leq 0.06$, $2.0 \text{ t/m}^3 \leq \rho \leq 3.1 \text{ t/m}^3$
5) For the 8 test run with depth-limited waves, breaking conditions were limited to spilling breakers which are not as damaging as plunging breakers. Therefore Eq (F.4) and Eq (F.5) may not be conservative in some breaking wave conditions.

Uncertainty of the formula: The coefficients of variation on the factor 6.2 in Eq (F.4) and on the factor 1.0 in Eq (F.5) are estimated to be 6.5% and 8%, respectively.

F.3 van der Meer (1990), submerged breakwaters

The following formula was established for submerged breakwaters with two-layer armour on front, crest and rear slope. Irregular, head-on waves.

$$\frac{H_c}{h} = (2.1 + 0.1S)\exp\left(-0.14 N_s^*\right) \qquad \text{Eq (F.6)}$$

where h water depth
- H_c height of structure over sea bed level
- S relative eroded area
- N_s^* spectral stability number by Ahrens (1987), $N_s^* = \frac{H_s}{\Delta D_{n50}} s_{0p}^{-1/3}$

The uncertainty of Eq (F.6) can be expressed by considering the factor 2.1 as a Gauss distributed stochastic variable with the mean 2.1 and a standard deviation of 0.35, i.e. a coefficient of variation of 17%.

The formula is based on tests by Givler and Sorensen (1986): Regular head-on waves, slope 1:1.5, and tests by Van der Meer (1988): Irregular head-on waves, slope 1:2

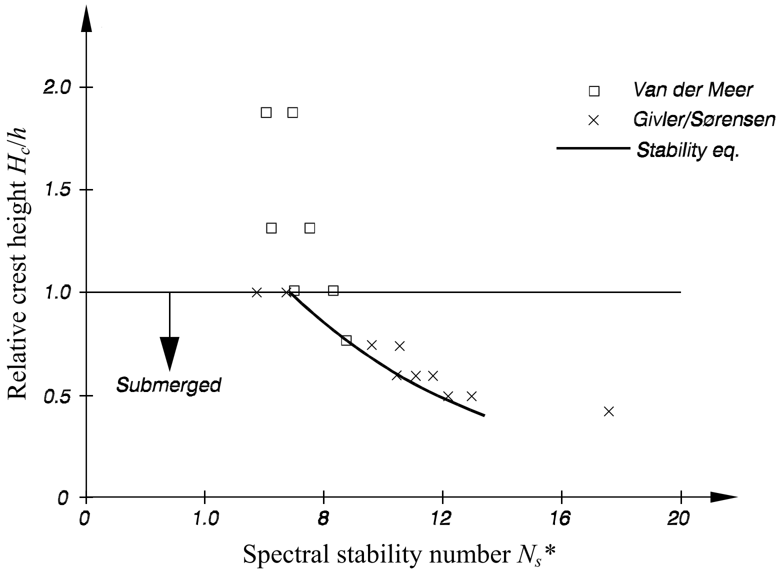


Figure F.1. Van der Meer (1990) formula and test data.

F.4 Vidal *et al.* (1992, 1995, 2000), head and trunk stability

Vidal *et al.* (1992) performed 3D small scale laboratory tests at NRC, Canada and proposed stability graphs for trunk corresponding to initiation of damage. In 1995 stability graphs for head damage for different damage levels were proposed. In 2000 a general methodology to calculate stability of LCS's was proposed and parameterized stability curves for initiation of damage were given for all structural sections. The tests are described in more detail in Appendix F.

Vidal *et al.* (1992) proposed the stability curves in Figure F.2 to be used for LCS trunks, and Vidal *et al.* (1995) proposed the stability curves in Figure F.3 to be used for LCS roundheads.

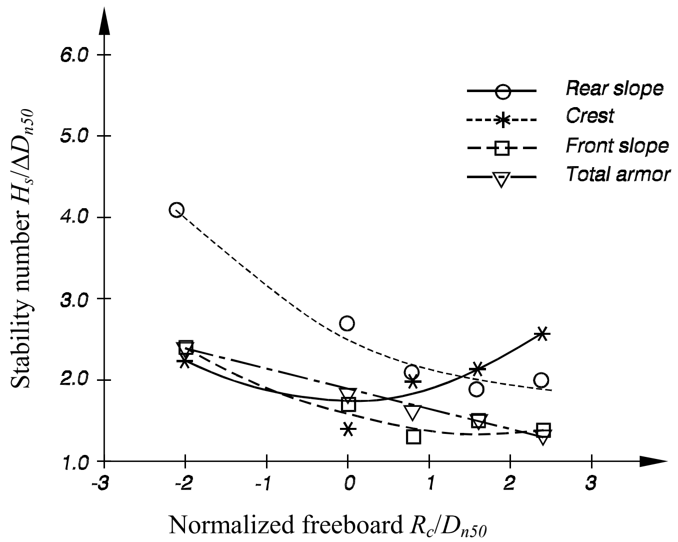


Figure F.2. Stability of trunk for initiation of damage according to Vidal *et al.* (1992).

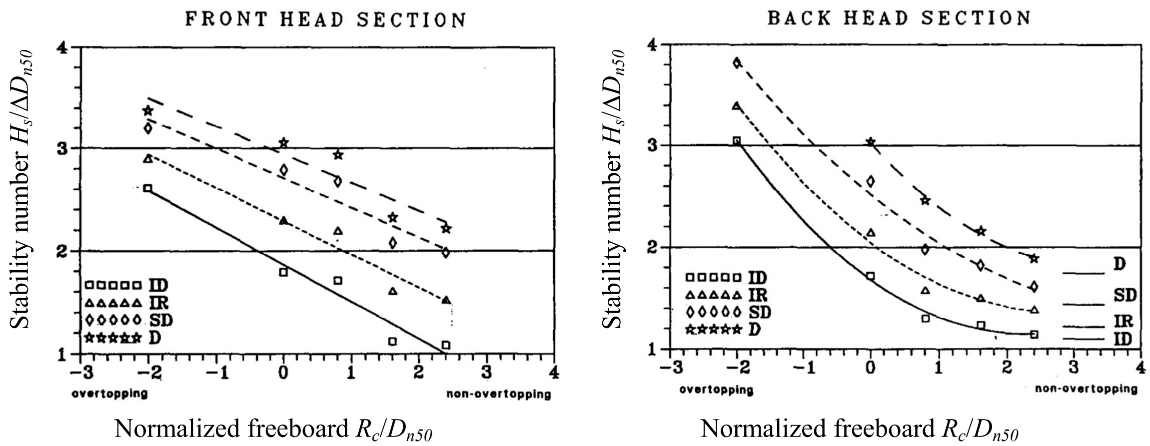


Figure F.3. Breakwater head stability curves for different levels of damage, Vidal *et al.* (1995). Damage level: ID, Initiation of Damage; IR, Iribarren's damage; SD, Start of Destruction; D, Destruction.

Vidal *et al.* (2000) proposed a more general methodology to evaluate stability for various low-crested breakwater geometries. Reduction factors were introduced including existing knowledge about conventional breakwaters. Parameterized curves corresponding to initiation of damage of trunk and head were given by Eq (F.7).

$$N_s = A + B\frac{R_c}{D_{n50}} + C\left(\frac{R_c}{D_{n50}}\right)^2 \quad , -2.01 < R_c/D_{n50} < 2.41 \qquad \text{Eq (F.7)}$$

Table F.2. Values of coefficients *A*, *B* and *C* in Eq (F.7).

Sector	*A*	*B*	*C*
Front slope and front head	1.831	-0.2450	0.0119
Crest	1.652	0.0182	0.1590
Back slope	2.575	-0.5400	0.1150
Back head	1.681	-0.4740	0.1050

Appendix G Existing armour stability data

G.1 UCA, 2001

Tests were carried out at the wave flume of the University of Cantabria (68.9 x 2 x 2 m). The cross section given in Table G.1 was tested in regular waves (52 tests) and irregular waves (16 tests). However, the stone size of the armour indicates that viscous scale effects were present in the tests. No references about the tests exist. A document describing the tests and digital data has been provided by Cesar Vidal, UCA.

Table G.1. Test conditions in UCA 2001 tests.

Number of tests	16 with irregular waves
Structure height	0.25 m
Crest width	0.25 m
Structure slope	1V:2H
Foreshore slope	1:20
Water depth	0.20 m and 0.30 m
Freeboard	-0.05 m and +0.05 m
Type of breakwater	Reef type
Materials	Quarry crushed limestone, W_{50} = 4.3 g, D_{n50} = 0.012 m
H_s	0.02 m to 0.07 m
T_p	1.8 sec to 3.4 sec
Test duration	1 hour (1300 to 2400 waves)

Table G.2. UCA 2001 test results for irregular waves.

TEST	$H_{1/3}$	$H_{1/10}$	$H_{1/100}$	H_{max}	T_p	R_c	S
9	0.022	0.029	0.041	0.058	1.8	0.05	0.70
10	0.042	0.057	0.080	0.093	1.8	0.05	9.17
20	0.053	0.073	0.102	0.126	2.6	0.05	50.70
21	0.039	0.054	0.077	0.099	2.6	0.05	1.36
29	0.043	0.058	0.081	0.096	3.4	0.05	8.47
30	0.033	0.044	0.063	0.101	3.4	0.05	2.00
38	0.036	0.047	0.055	0.085	1.8	-0.05	0.89
39	0.062	0.081	0.096	0.153	1.8	-0.05	3.68
46	0.044	0.058	0.067	0.091	2.2	-0.05	0.87
47	0.061	0.081	0.096	0.153	2.2	-0.05	4.22
52	0.054	0.072	0.082	0.131	2.6	-0.05	2.02
53	0.038	0.050	0.057	0.089	2.6	-0.05	1.04
58	0.065	0.088	0.098	0.146	3.0	-0.05	8.59
59	0.036	0.048	0.047	0.085	3.0	-0.05	0.99
67	0.074	0.098	0.108	0.157	3.4	-0.05	11.85
68	0.041	0.054	0.058	0.107	3.4	-0.05	0.01

$H_{1/n}$ Average of the highest *1/n* incident waves, in metres
H_{max} Maximum incident wave height in metres
T_p Incident peak period in seconds
R_c Freeboard in metres
S Damage according to Broderick (1983)

To investigate the influence of wave period all data are plotted in Figure G.1. It is seen that all data follows the same trend in damage progress.

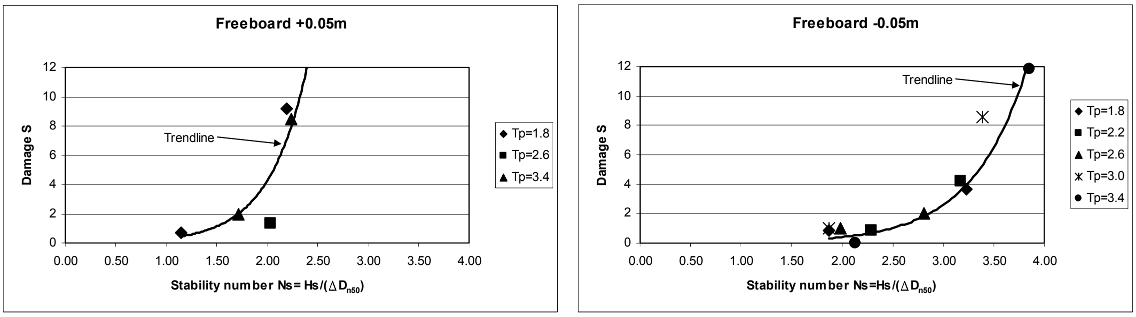


Figure G.1. Damage in the UCA 2001 tests.

The data have been used to decide on the stone size to be used in the new 3-D tests at AAU, but due to the viscous scale effects the tests have not been used further.

G.2 Delft, 1995

Burger (1995) tested the influence of rock shape and grading on the stability of front, crest and rear slope of low-crested structures. No or very small influences were found. Tests were performed at Delft Hydraulics in the "Shelde basin" at the "De Voorst". Results are presented by Burger (1995) and Van der Meer *et al.* (1996).

Table G.3. Test details for Delft 1995 tests.

Number of tests	76
Structure height	0.67 m
Crest width	?
Structure slope	Seaward 1:2, and rear 1:1.5
Foreshore slope	Horizontal
Water depth	0.6 m
Freeboard	+0.07 m
Type of breakwater	2 layer conventional type
Materials	Rock D_{n50} = 0.035 m
H_s	0.07 m to 0.18 m
T_p	Two different steepness' s_p = 0.02 and s_p = 0.04. T_p = 1.5 s to 2.4 s
Test duration	1000 waves

Table G.4. Results from Delft 1995 tests.

serie	test	H_s *(m)*	D_{n50} *(m)*	ρ (kg/m^3)	T_p *(s)*	*s*	*S* front	*S* crest	*S* rear
1a	1	0.071	0.0351	2700	1.586	0.02	0.00	0.51	0.46
	2	0.098	0.0351	2700	1.7986	0.02	0.85	0.48	0.54
	3	0.124	0.0351	2700	2.0276	0.02	1.41	0.22	1.33
	4	0.144	0.0351	2700	2.1686	0.02	3.08	1.31	2.08
	5	0.170	0.0351	2700	2.3844	0.02	5.70	2.75	3.42
	6	0.178	0.0351	2700	2.4292	0.02	4.27	1.84	5.33
1b	1	0.082	0.0351	2700	1.1382	0.04	0.00	0.00	0.00
	2	0.103	0.0351	2700	1.2822	0.04	0.95	0.00	0.00
	3	0.123	0.0351	2700	1.3744	0.04	1.52	0.12	0.00
	4	0.145	0.0351	2700	1.5008	0.04	3.61	0.00	0.00
	5	0.161	0.0351	2700	1.6106	0.04	6.05	1.05	0.74
	6	0.182	0.0351	2700	1.7194	0.04	11.55	1.00	2.74
2a	1	0.071	0.0347	2700	1.586	0.02	0.00	0.00	0.49
	2	0.098	0.0347	2700	1.7986	0.02	1.32	0.44	1.00
	3	0.124	0.0347	2700	2.0276	0.02	3.32	0.85	0.86
	4	0.144	0.0347	2700	2.1686	0.02	5.65	0.53	2.57
	5	0.170	0.0347	2700	2.3844	0.02	10.04	3.76	3.99

serie	test	H_s (m)	D_{n50} (m)	ρ (kg/m^3)	T_p (s)	s	S front	S crest	S rear
	6	0.178	0.0347	2700	2.4292	0.02	32.22	24.79	12.19
2b	1	0.082	0.0347	2700	1.1382	0.04	0.15	0.00	0.00
	2	0.103	0.0347	2700	1.2822	0.04	1.06	0.02	0.00
	3	0.123	0.0347	2700	1.3744	0.04	1.15	0.00	0.00
	4	0.145	0.0347	2700	1.5008	0.04	5.13	0.00	0.00
	5	0.161	0.0347	2700	1.6106	0.04	7.72	0.11	0.26
	6	0.182	0.0347	2700	1.7194	0.04	12.52	2.07	0.00
3a	1	0.078	0.0335	2700	1.6292	0.02	0.00	0.32	0.64
	2	0.101	0.0335	2700	1.7758	0.02	2.22	0.25	0.00
	3	0.119	0.0335	2700	1.9896	0.02	3.36	2.43	0.56
	4	0.140	0.0335	2700	2.0936	0.02	7.30	1.25	4.19
	5	0.160	0.0335	2700	2.3282	0.02	11.93	4.33	1.75
	6	0.184	0.0335	2700	2.4296	0.02	25.67	26.42	12.00
3b	1	0.084	0.0335	2700	1.1338	0.04	0.43	0.00	0.00
	2	0.102	0.0335	2700	1.2746	0.04	1.22	0.36	0.00
	3	0.120	0.0335	2700	1.3778	0.04	2.39	0.87	2.09
	4	0.143	0.0335	2700	1.4954	0.04	6.70	0.00	0.00
	5	0.160	0.0335	2700	1.6082	0.04	6.36	0.47	0.11
	6	0.181	0.0335	2700	1.71	0.04	6.61	1.99	5.83
4a	1	0.078	0.0338	2700	1.6292	0.02	0.54	0.00	0.00
	2	0.101	0.0338	2700	1.7758	0.02	2.22	0.09	1.40
	3	0.119	0.0338	2700	1.9896	0.02	3.62	1.37	0.21
	4	0.140	0.0338	2700	2.0936	0.02	7.07	1.49	5.76
	5	0.160	0.0338	2700	2.3282	0.02	11.39	1.02	1.69
	6	0.184	0.0338	2700	2.4296	0.02	10.98	18.43	7.04
4b	1	0.084	0.0338	2700	1.1338	0.04	0.00	0.00	0.33
	2	0.102	0.0338	2700	1.2746	0.04	0.97	0.00	0.00
	3	0.120	0.0338	2700	1.3778	0.04	1.38	0.08	0.23
	4	0.143	0.0338	2700	1.4954	0.04	5.18	0.02	0.49
	5	0.160	0.0338	2700	1.6082	0.04	4.72	0.15	1.74
	6	0.181	0.0338	2700	1.71	0.04	12.75	1.40	0.73
5a	1	0.059	0.0336	2700	1.3736	0.02	0.00	0.00	0.00
	2	0.080	0.0336	2700	1.6426	0.02	1.95	0.24	0.00
	3	0.102	0.0336	2700	1.7788	0.02	1.42	0.50	0.59
	4	0.120	0.0336	2700	2.0146	0.02	3.10	0.49	0.00
	5	0.142	0.0336	2700	2.1218	0.02	9.30	1.09	3.43
	6	0.160	0.0336	2700	2.338	0.02	8.10	3.32	4.09
	7	0.188	0.0336	2700	2.4038	0.02	14.80	5.16	5.63
5b	1	0.059	0.0336	2700	0.9768	0.04	0.20	0.22	0.22
	2	0.081	0.0336	2700	1.1354	0.04	0.67	0.00	0.00
	3	0.101	0.0336	2700	1.282	0.04	1.31	0.00	0.00
	4	0.122	0.0336	2700	1.3746	0.04	1.38	0.20	1.10
	5	0.143	0.0336	2700	1.5046	0.04	5.69	0.08	0.40
	6	0.160	0.0336	2700	1.6096	0.04	8.03	0.28	0.44
	7	0.182	0.0336	2700	1.6924	0.04	15.17	0.89	1.36
6a	1	0.059	0.0368	2550	1.3736	0.02	0.27	0.06	0.03
	2	0.080	0.0368	2550	1.6426	0.02	0.16	0.05	0.12
	3	0.102	0.0368	2550	1.7788	0.02	1.65	1.62	0.56
	4	0.120	0.0368	2550	2.0146	0.02	6.02	2.63	3.29
	5	0.142	0.0368	2550	2.1218	0.02	8.57	7.11	3.89
	6	0.160	0.0368	2550	2.338	0.02	>50	>50	>50
	7	0.188	0.0368	2550	2.4038	0.02	>50	>50	>50
6b	1	0.059	0.0368	2550	0.9768	0.04	0.10	0.04	0.00

serie	test	H_s (m)	D_{n50} (m)	ρ (kg/m^3)	T_p (s)	s	S front	S crest	S rear
	2	0.081	0.0368	2550	1.1354	0.04	0.12	0.19	0.00
	3	0.101	0.0368	2550	1.282	0.04	1.14	0.00	0.00
	4	0.122	0.0368	2550	1.3746	0.04	2.84	0.50	0.12
	5	0.143	0.0368	2550	1.5046	0.04	4.95	0.90	0.00
	6	0.160	0.0368	2550	1.6096	0.04	10.58	4.12	1.70
	7	0.182	0.0368	2550	1.6924	0.04	>50	>50	>50

G.3 NRC, 1992

Vidal *et al.* (1992) performed 3D small scale laboratory tests at NRC, Canada. Details on setup are found in Vidal *et al.* (1995) and in Table G.5. Tests were performed on a complete 3D structure, and damage was measured in trunk and roundhead. The trunk was divided in front slope (FS), back slope (BS), crest (C), and total slope (TS). The roundhead was divided in front head (FH) covering an area of 60° in the seaward part, and back head (BH), which covered the remaining 120° of the leeward part of the roundhead.

Table G.5. Test details for NRC 1992 tests.

Number of tests	35
Structure length	4.7 m
Structure height	40 cm and 60 cm
Crest width	0.15 m ($6 \cdot D_{n50}$)
Structure slope	1:1.5
Foreshore slope	Horizontal bed
Water depth	38 cm to 65 cm
Freeboard	-0.05, 0.0, 0.02, 0.04, 0.06
Type of breakwater	2 layer conventional type
Materials	Gravel armour: D_{n50} = 2.5 cm, D_{85}/D_{15} = 1.1, ρ_s = 2650 kg/m^3, n = 0.44
	Gravel core: D_{n50} = 1.9 cm, D_{85}/D_{15} = 1.4, ρ_s = 2650 kg/m^3, n = 0.44
H_s	0.05 m to 0.15 m
T_p	1.4 s and a few 1.8 s

Damage *S* to the trunk was calculated according to Broderick (1983), but for the roundhead the methodology described in Vidal *et al.* (1995) was used, see Chapter 3.

Table G.6. Results of NRC 1992 tests. GD is degree of damage (see Chapter 3.5).

TEST	H_s	T_p	R_c	*S*(BH)	GD	*S*(FH)	GD	*S*(TS)	GD	*S*(C)	GD	*S*(BS)	GD	*S*(FS)	GD
1	0.047	1.39	0	0.39	ND	0.39	ND	0.45	ND	0.72	ND	0	ND	0.45	ND
4	0.073	1.4	0	1.97	ID	0	ND	1.27	ND	1	ID	0.09	ND	0.81	ID
5	0.073	1.4	0	0.98	ID	0.66	ND	2.08	ID	1.09	ID	0.18	ND	0.36	ND
2	0.092	1.41	0	2.38	IR	2.38	ID	4.74	IR	4.64	IR	0.45	ND	2.87	IR
3	0.11	1.41	0	3.47	IR	3.73	IR	5.15	IR	2.97	IR	0.18	ND	3.31	IR
13	0.126	1.4	0	12.12	D	13.73	D	17.61	D	9.83	SD	0.82	ID	9.19	D
9	0.074	1.39	-0.05	0	ND	0	ND	0	ND	0.18	ND	0	ND	0.27	ND
6	0.086	1.41	-0.05	0	ND	0.4	ND	1.63	ID	1.36	ID	0.09	ND	0.63	ND
7	0.112	1.41	-0.05	0.51	ND	2.4	ID	2.53	IR	2.72	IR	0.18	ND	1.81	ID
8	0.124	1.41	-0.05	0.64	ND	1.93	ID	4.54	IR	2.44	IR	0.27	ND	4.21	SD
14	0.132	1.4	-0.05	1.42	ID	10.13	D	5.35	IR	4.6	IR	0.09	ND	2.72	IR
15	0.152	1.41	-0.05	3.3	IR	14.33	D	10.72	SD	10.22	SD	0.54	ND	5.03	SD
16	0.054	1.4	0.02	1.16	ND	0	ND	1.36	ND	0.09	ND	0.18	ND	0.27	ND
12	0.073	1.41	0.02	3.55	IR	1.48	ID	3.54	IR	1.09	ID	0.27	ND	2.44	IR
10	0.092	1.41	0.02	6.35	SD	1.63	ID	6.43	IR	1.27	ID	0.27	ND	4	SD
11	0.103	1.41	0.02	8.62	SD	5.34	IR	8.8	SD	3.57	IR	0.63	ND	5.31	SD
17	0.146	1.4	0.02	15.38	D	23.18	D	43.76	D	8.63	SD	1.54	ID	11.83	D
18	0.045	1.41	0.04	0.71	ND	0.85	ND	0.45	ND	0.09	ND	0	ND	0.27	ND
19	0.077	1.4	0.04	4.36	SD	3.68	IR	4.13	IR	1.27	ID	0.36	ND	2.78	IR
20	0.094	1.4	0.04	12.18	D	13.78	D	6.68	SD	1.99	ID	0.54	ND	4.72	SD
21	0.116	1.4	0.04	18.24	D	19.14	D	22.31	D	2.62	ID	1.27	ID	11.22	D
22	0.136	1.41	0.04	-	-	-	-	-	-	4.76	IR	0.91	ID	-	-
23	0.151	1.41	0.04	-	-	-	-	-	-	3.28	IR	3.17	IR	-	-
24	0.052	1.41	0.06	1.41	ID	1.41	ID	1.09	ND	0	ND	0	ND	0.91	ID
25	0.077	1.42	0.06	8.32	SD	5.1	IR	4.08	IR	0.18	ND	0	ND	2.98	IR
26	0.09	1.41	0.06	16.43	D	8.92	SD	5.16	IR	0.36	ND	1	ND	5.62	SD
27	0.109	1.41	0.06	-	-	12.58	D	19.56	D	1.18	ID	2.26	IR	11.09	D
28	0.122	1.41	0.06	-	-	-	-	-	-	1.81	ID	3.08	IR	-	-
29	0.132	1.41	0.06	-	-	-	-	-	-	2.35	IR	3.3	SD	-	-
30	0.05	1.82	0.02	0.51	ND	0	ND	0.91	ND	0.36	ND	0	ND	0.36	ND
31	0.078	1.82	0.02	4.22	SD	4.64	IR	4.7	IR	1.81	ID	0.45	ID	3.54	IR
32	0.105	1.81	0.02	13.28	D	11.63	D	16.49	D	3.69	IR	0.54	ND	6.79	SD
33	0.131	1.81	0.02	-	-	16.84	-	-	-	3.7	IR	0.27	ND	16.12	D
34	0.07	1.82	0.06	2.48	IR	0.91	ND	4.46	IR	0.27	ND	0	ND	2.99	IR
35	0.096	1.82	0.06	15.59	D	6.44	SD	26.55	D	0.36	ND	0.72	ND	8.87	D

G.4 Delft, 1988

Van der Meer (1988) performed 31 LCS stability tests in the wave flume (1.0 m wide, 1.2 m deep and 50 m long) at Delft Hydraulics. All tests were performed with 1000 waves and 3000 waves. Water depth was kept constant and structure height was varied.

Table G.7. Test details for Delft 1988 tests.

Number of tests	31
Structure height	0.30 m, 0.40 m and 0.52 m
Crest width	$8D_{n50}$
Structure slope	1:2
Foreshore slope	1:30
Water depth	0.4 m
Freeboard	-0.1 m, 0, +0.125 m
Type of breakwater	Two layer conventional type
Materials	Armour D_{n50} = 0.0344 m, Core D_{n50} = 0.019 m. ρ_s = 2600 kg/m^3
H_s	0.08 m to 0.22 m
T_p	1.96 sec and 2.56 sec
Test duration	Test with both 1000 and 3000 waves

Table G.8. Selected results from Delft 1988 tests.

Test	Structure height (m)	T_p (s)	H_s (m)	$H_s/\Delta D_{n50}$ (-)	T_m (s)	Damage S (N_z = 1000)	(N_z = 3000)
PA001	0.4	1.96	0.105	1.9077	1.70	1.480	2.47
PA002	0.4	1.96	0.125	2.27108	1.72	4.200	4.40
PA003	0.4	1.98	0.145	2.63445	1.72	2.870	8.63
PA004	0.4	1.96	0.174	3.16134	1.72	13.530	20.54
PA005	0.4	1.96	0.083	1.50799	1.70	1.280	1.72
PA006	0.4	2.56	0.134	2.43459	2.21	3.850	4.66
PA007	0.4	2.56	0.159	2.88881	2.22	3.520	5.52
PA008	0.4	2.56	0.196	3.56105	2.19	16.910	46.38
PA009	0.4	2.56	0.111	2.01672	2.22	2.010	2.92
PA010	0.4	2.53	0.077	1.39898	2.21	0.860	1.02
PA011	0.4	2.56	0.176	3.19767	2.21	9.620	17.87
PA012	0.525	2.60	0.137	2.4891	2.21	3.270	5.64
PA013	0.525	2.60	0.162	2.94331	2.20	13.040	21.98
PA014	0.525	2.56	0.112	2.03488	2.19	3.050	3.39
PA015	0.525	2.50	0.078	1.41715	2.21	0.680	0.75
PA016	0.525	2.56	0.149	2.70712	2.22	8.660	14.54
PA017	0.525	1.94	0.128	2.32558	1.70	6.690	12.27
PA018	0.525	1.96	0.105	1.9077	1.68	2.450	3.54
PA019	0.525	1.94	0.083	1.50799	1.68	1.160	1.84
PA020	0.525	1.96	0.148	2.68895	1.70	14.070	45.86
PA021	0.3	1.96	0.147	2.67078	1.72	1.590	2.53
PA022	0.3	1.94	0.175	3.17951	1.72	4.640	7.02
PA023	0.3	1.96	0.196	3.56105	1.72	4.630	6.77
PA024	0.3	1.96	0.216	3.92442	1.74	10.100	13.54
PA025	0.3	1.94	0.116	2.10756	1.70	1.450	1.71
PA026	0.3	1.98	0.161	2.92515	1.72	1.810	2.05
PA027	0.3	2.53	0.193	3.50654	2.18	7.660	11.60
PA028	0.3	2.56	0.161	2.92515	2.18	4.230	7.43
PA029	0.3	2.56	0.137	2.4891	2.18	2.000	3.11
PA030	0.3	2.56	0.11	1.99855	2.18	0.970	1.20
PA031	0.3	2.60	0.219	3.97892	2.16	13.470	16.96

Appendix H Wave transmission

Based on Burcharth and Kramer (2002) the most used formulae in designing of LCS's for coast protection purposes is initially presented in Chapter H.1. It has not been the intention to make a comprehensive literature review of all existing knowledge, as it would require a long detailed report. Instead a brief summary of a few design formulae is given. The formulae are all developed by use of wave channel tests with irregular, head-on waves. As LCS's can be exposed to oblique wave attack an experimental investigation of 3D effects especially the wave obliquity was performed within DELOS at Aalborg University. Based on Kramer *et al.* (2005) these tests are briefly described in Chapter H.2.

H.1 Existing formulae for wave transmission

Values of the transmission coefficient given in the literature are almost all from laboratory experiments, many of which were conducted at rather small scales. Some scale effects might influence the results, especially for the proportion related to wave penetration. Detached breakwaters for coastal protection are placed in shallow water and are often built entirely of armour blocks, i.e. without underlayer and core. Such breakwaters are very porous giving high wave transmission by wave penetration. Caution should therefore be taken when using the following design formulae for such structures.

H.1.1 Powell and Allsop (1985)

Powell and Allsop (1985) investigated the influence of freeboard on the wave transmission coefficient, see Figure H.1. A multilayer structure was tested at emerging conditions, and a homogeneous structure was tested at submerged and slightly emerged conditions.

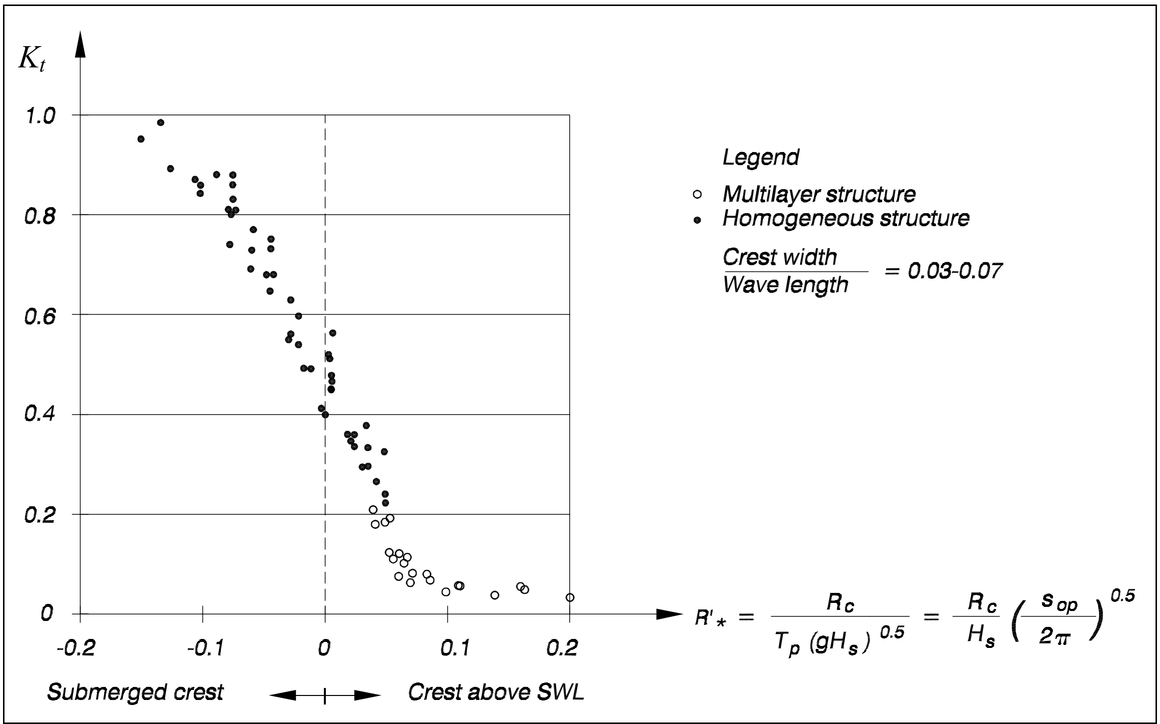


Figure H.1. Wave transmission coefficient K_t by Powell and Allsop (1985).

H.1.2 Van der Meer and d'Angremond (1991)

Van der Meer and d'Angremond (1991) developed the wave transmission formula given by Eq (H.1). The database included tests with conventional rock armoured low-crested breakwaters and reef breakwaters subject to both submerged and emerging conditions.

$$K_t = \left(0.031\frac{H_s}{D_{n50}} - 0.24\right)\frac{R_c}{D_{n50}} + b \qquad \text{Eq (H.8)}$$

where

$$b = \begin{cases} -5.42 s_{0p} + 0.0323\frac{H_s}{D_{n50}} - 0.0017\left(\frac{B}{D_{n50}}\right)^{1.84} + 0.51 & \text{conventional structure} \\ -2.6 s_{0p} - 0.05\frac{H_s}{D_{n50}} + 0.85 & \text{reef type structure} \end{cases}$$

Limits:

$K_{t,max} = 0.75$, $K_{t,min} = 0.075$ conventional structure

$K_{t,max} = 0.60$, $K_{t,min} = 0.15$ reef type structure

Tested ranges : $1 < H_s/D_{n50} < 6 \qquad 0.01 < s_{0p} < 0.05 \qquad -2 < R_c/D_{n50} < 6$

Figure H.2 shows an example of the use of Eq (H.8).

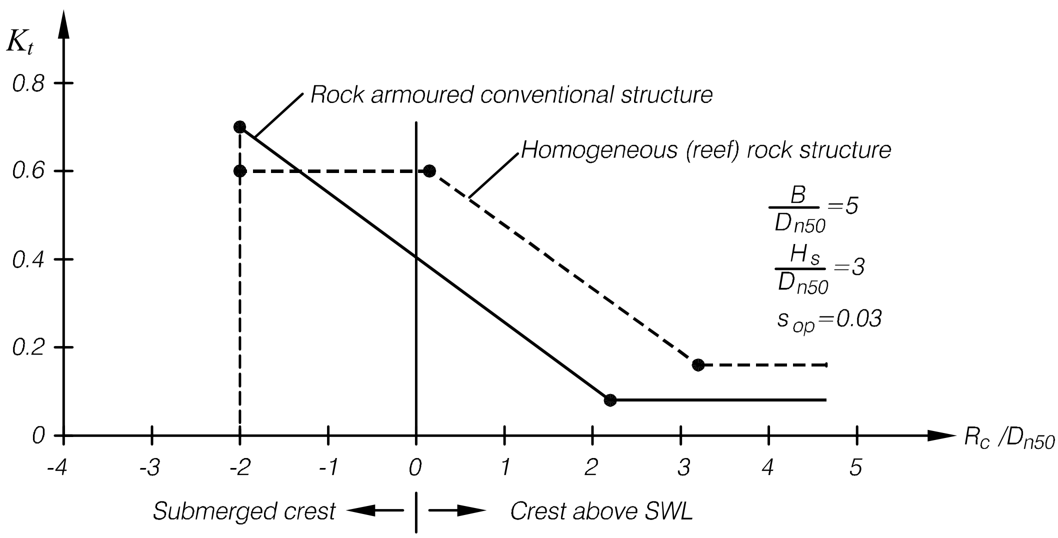


Figure H.2 Example of wave transmission coefficients calculated by Eq (H.8).

H.1.3 Sand Jensen (2002)

Sand Jensen (2002) performed small scale wave flume tests with submerged reefs subject to irregular head-on waves as shown in Figure H.3. The wave transformation from the crest of the reef and along the reef-plateau was recorded. Eq (H.2) is developed for reefs, but it may also be used in case of extremely wide submerged structures.

$$K_t = \sqrt{\left(\frac{-0.36R_c}{H_s}\right) + \tanh\left(0.06\left[\frac{B}{L_t}\right]^{-1.5}\right)}$$

Eq (H.9)

Limits:

Normalized off-reef wave height ($-H_s/R_c$) $\geq$ 0.36 to ensure wave breaking

Distance from reef-crest (B/L_t) $\geq$ 0.3 to ensure initiation of wave breaking

Table H.1. Tested ranges of parameters in Eq (H.2).

R_c [m]	L_t [m]	H_s/R_c [-]	s_{op}[-]	$H_s/(gT_p^2)$ [-]
-0.205 to -0.275	2.7 to 6.2	-0.36 to -1.02	0.016 to 0.078	0.0015 to 0.0109

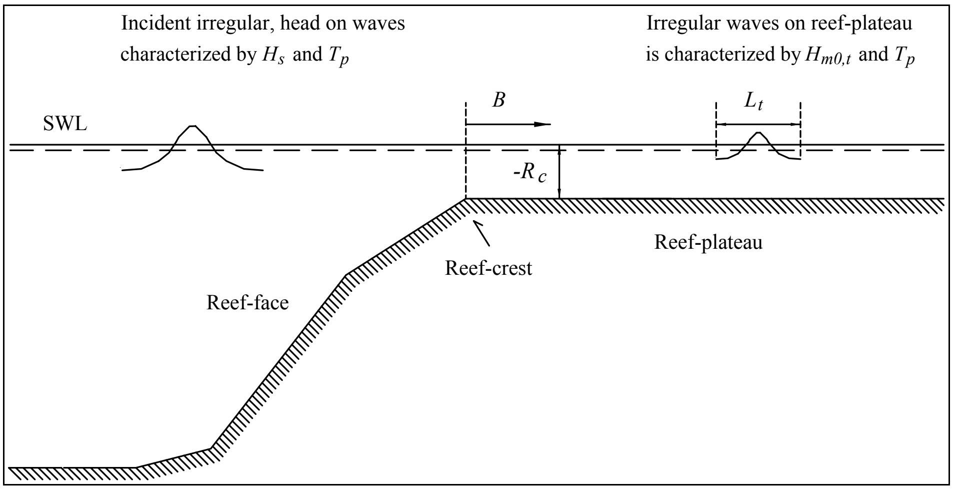


Figure H.3. Notation for wave transmission formula by Sand Jensen (2002).

An important outcome of the study by Sand Jensen (2002) was, that no significant influence on the wave transmission was observed within the tested range of inclinations of the reef (1:0.5-1:2). This indicates, that for low crested breakwaters the effect of structure slope on wave transmission characteristics presumably is marginal.

H.2 Three-dimensional wave transmission tests at AAU 2002

In continuation of the DELOS stability tests described in Appendix D wave transmission tests were performed in the short-crested wave basin at Aalborg University described in Chapter D.1. The test setups are described in detail in Kramer *et al.* (2003), and a brief summary is given in Kramer *et al.* (2005), from which the main part of following text has been copied. Some errors in the text and analysis with respect to the wave transmission tests are present in Kramer *et al.* (2003). The author has corrected these errors for the rubble mound tests, and the author is confident that the analysis and text presented in the following is correct. However, the new analysis does not change the original conclusions notably.

The main goal of the new tests was to investigate the influence of oblique waves on the wave transmission. 168 tests were performed with the objective of studying influences of wave obliquities on transmitted wave energy, wave directions and spectral changes. 3 structural set-ups with rubble structures and 3 set-ups with smooth plywood structures were tested in 2D and 3D waves.

The author had the structures built in the laboratory, performed the tests, and made a great deal of the reporting and analysis. Several Italian DELOS participants assisted in this work, particularly Barbara Zanuttigh and Matteo Tirindelli (University of Bologna) and Marcello di Risio (3rd University of Rome). However, guidelines and new formulae established from the tests were developed mainly by Baoxing Wang (University of Plymouth), Riccardo Briganti (University of Rome), Jentsje van der Meer (INFRAM), and Barbara Zanuttigh. These people are also acknowledged for their help in the planning and/or execution of the tests. Results and guidelines elaborated from the wave transmission tests can be found in Van der Meer *et al.* (2003, 2005).

H.2.1 Layouts and cross sections

Two structures were tested: A rubble mound structure and a smooth structure made out of plywood. Three layouts were constructed for each structure: 0° (perpendicular wave attack, structure parallel with the wave generator), 30° and 50°. Figure H.4 shows the layouts for rubble structure inclined of 30° with respect to the beach (top) and smooth structure (bottom). The schemes for the whole set of layouts are presented in Figure H.5 (rubble structure) and Figure H.6 (smooth structure). The rubble mound structure was 25 cm high with a crest width of 10 cm and it was built of quarry rock. The cross-section consisted of a bottom layer, a core and an outer armour layer with the detailed characteristics: $W_{50} = 0.269$ kg, $D_{n50} = 0.0466$ m and a grading of $D_{85}/D_{15} = 1.25$, see the cross section scheme at the top and left-hand side of Figure H.5. The smooth structure had gentler slopes than the rubble mound structure, which is also the case in reality. The seaward slope was 1:3 and the leeward slope 1:2. The structure height was 0.30 m and the crest width 0.20 m.

The structures were placed on a horizontal plateau, which was 0.16 m higher than the bottom of the basin. This created a larger depth in front of the wave generator and made it possible to generate very steep and breaking waves in front of the structure, see Figure H.7. Reflection from the rear wall of the basin was minimised using a 1:5 rubble beach.

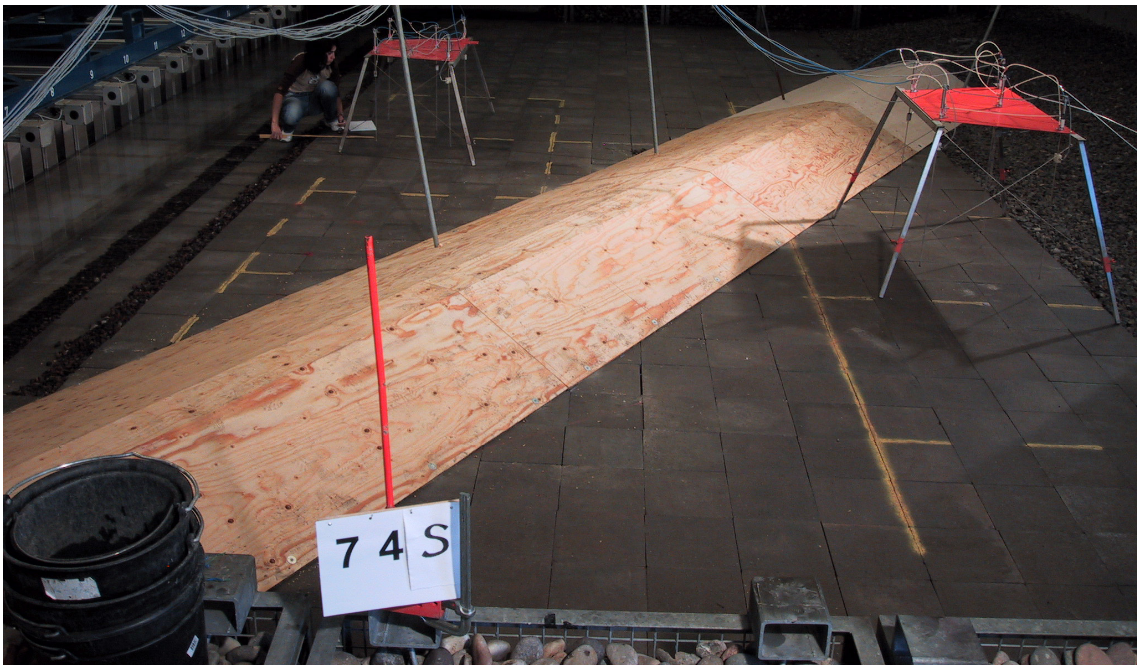


Figure H.4. Views of the basin, 30° tests on rubble and smooth plywood structure.

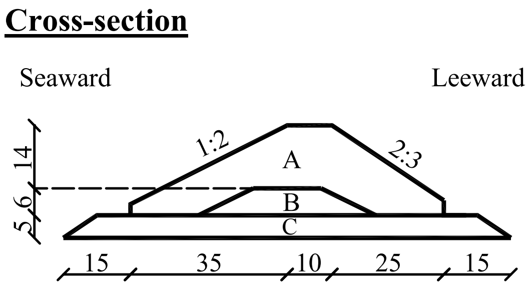
H.2.2 Measurements

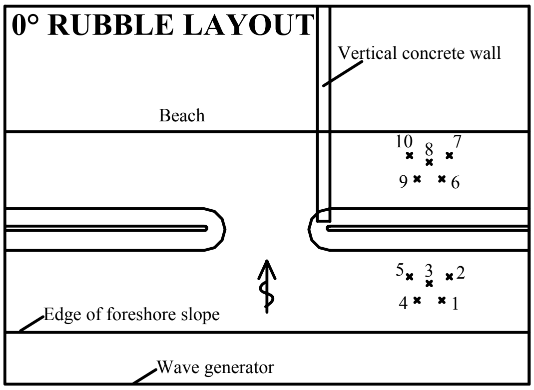
For both the rubble mound structure as well as for the smooth structure 84 tests were performed. Table H.2 gives an overall view. Three crest freeboards were tested with two wave steepness' and three wave heights, giving 18 conditions for each wave direction. The main angles of wave attack were 0°, 30° and 50°, but as the multi-directional wave generator could also generate waves under an angle, a limited number of tests were performed with 20°, 40° and 60°. A Jonswap spectrum with peak enhancement factor equal to 3.3 was used for all the tests.

Only 10 of the 84 tests were performed with long-crested waves. The remaining 74 short-crested tests were performed with a $\cos^{2S_\theta}$ spreading function with $S_\theta = 50$. Incident and transmitted wave conditions were measured. A wave gauge array of five gauges was placed in front of the structure to measure the incident waves and a similar array behind the structure to measure the transmitted waves, see the placements in Figure H.5 and Figure H.6. Measurements from the five-gauge array were used to calculate the directional wave spectra.

A sampling rate of 40 Hz was used throughout the experiments. The recorded length of each test was 15 minutes. Digital video of three minutes and digital photos were taken for each test.

Reflections from the smooth structure were expected to be large. Generation of standing waves due to multi reflections between wave generator and structure in case of perpendicular wave attack were likely to occur. To minimize this effect the structure was inclined at 30°, and 30° waves were generated giving perpendicular wave attack. In this way a large part of the reflected waves could escape the area between the structure and the wave generator and get absorbed in the absorbing side walls shown in Figure D.1.





Layer	D_{n50}
Armour, A	0.0466
Core, B	0.031
Bedding layer, C	0.015

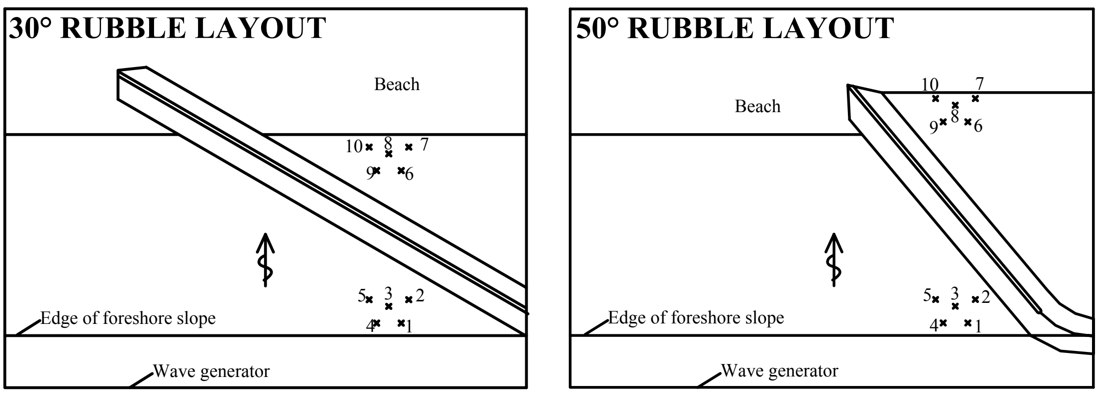
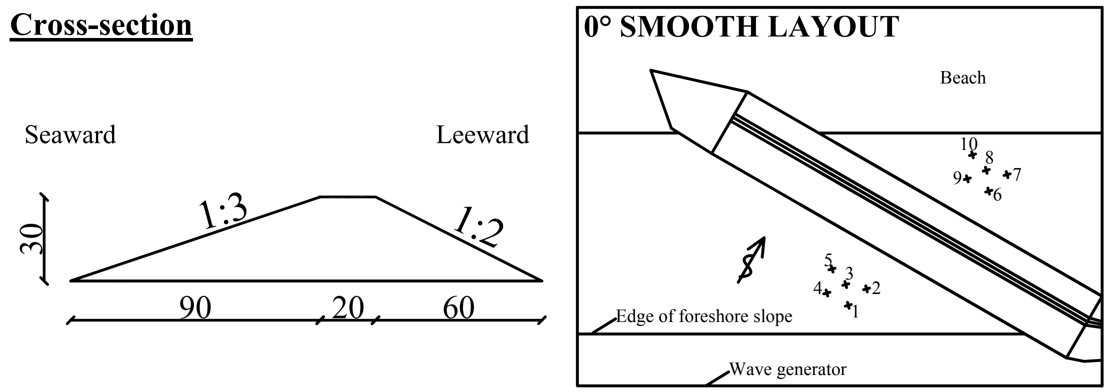


Figure H.5. Rubble structure layouts. 'X' marks the position of wave gauges.



Plywood structure, outer shape is shown.

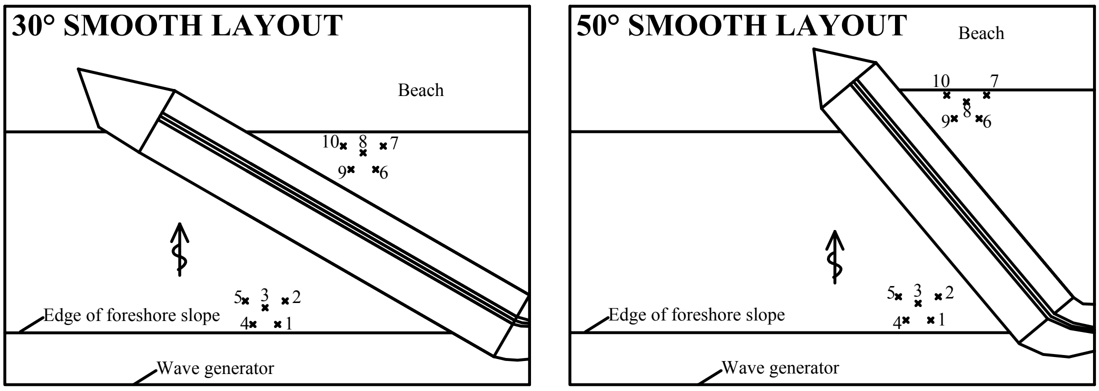


Figure H.6. Smooth structure layouts. 'X' marks the position of wave gauges.

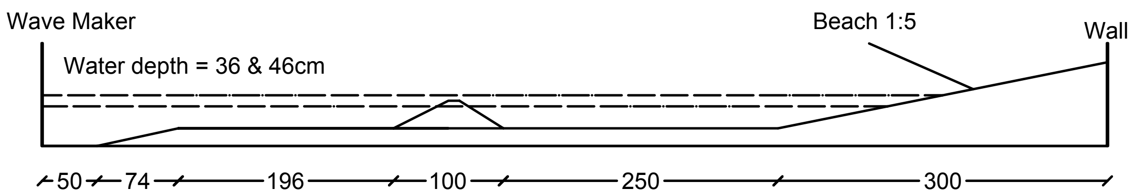


Figure H.7. Bottom topography for 0° rubble layout. Measures in cm.

Table H.2. Overall view of test program.

Tests per structure	84 (10 long-crested, 74 short-crested)
Crest freeboard	+0.05 m; 0.0 m; -0.05 m
Dimensionless freeboard R_c/H_s	-0.7 to +0.8
Wave height H_s	0.07 m to 0.14 m
Wave steepness s_{op}	0.02 and 0.04
Angles of wave attack β	0°, 20°, 30°, 40°, 50° and 60°

H.2.3 Some results with the rubble structure

Wave recordings from the two wave gauge arrays were used to estimate the directional wave spectrum seaward and leeward of the structure. The BDM method (Baysian Directional spectrum estimation Method) was used for this purpose. The directional spectrum was divided into incoming wave energy (energy in the interval of directions from -90° to +90°, with 0° as perpendicular wave attack), and reflected energy. Hereby the incoming and reflected significant wave heights, directional spreading, and the mean wave direction were calculated. In this way influences of reflections from structure and beach on the results were minimized. In order to check the capability of the 3D analysis to calculate the significant wave heights in case of long-crested waves, a comparison was made to 2D wave analysis by the Mansard and Funke (1980) method using three wave gauges, see Table H.3. The calculated significant wave heights by the two methods were approximately the same, and results of long-crested wave attack using 3D analysis is therefore correct. For more general details about wave analysis, see Chapter D.6. Detailed analyzed results using the BDM method are given in Chapter H.2.4.

Table H.3. Analysed waves in front of the structure in Test 08 (long-crested head on wave attack).

	3D analysis by BDM	**2D analysis by Mansard and Funke (1980) method**			
Wave gauges used	WG 1,2,3,4,5	WG 1,3,2	WG 1,3,5	WG 4,3,2	WG 4,3,5
H_{m0} incident [m]	0.13	0.12	0.12	0.12	0.12
H_{m0} reflected [m]	0.04	0.03	0.03	0.03	0.02
Reflection [%]	28	23	27	24	21

Wave conditions included shallow water waves ($H_{m0}/h \approx 0.3$) and heavy breaking depth limited waves ($H_{m0}/h \approx 0.6$) for all layouts and freeboards, see Table H.4.

Table H.4. Tested ranges of H_{m0}/h depending on layout (direction) and freeboard.

Freeboard R_c/H_c		-0.2 (submerged)	0 (zero freeboard)	+0.2 (emerged)
H_{m0}/h	Layout 1 (gap)	0.26 – 0.54	0.26 - 0.57	0.31 – 0.56
	Layout 2 (30°)	0.28 – 0.56	0.29 – 0.60	0.31 – 0.63
	Layout 3 (50°)	0.29 – 0.54	0.30 – 0.58	0.31 – 0.61

No active control of the water level set-up was performed in the experiments, and overtopping water could only escape over or through structures and through the porous boundaries. As no gaps were present, unrealistic set-ups developed during the experiments. It was important that these set-ups did not change the freeboard notably, as this could change the wave transmission significantly. The numerically largest mean set-ups developed during testing on Layout 1 (gap), and in this case the numerically largest mean set-up was less than just 4 % of the water depth, see Figure H.8. The influence of the set-up on the results is therefore marginal.

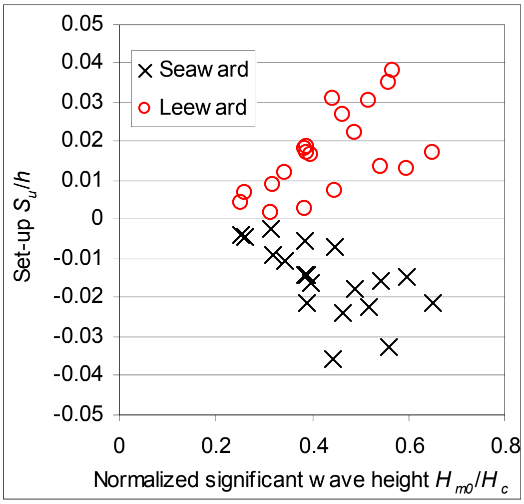


Figure H.8. Mean set-up in tests on layout 1 (test no. 1 to 20). Positive set-up is increase in water level.

All the test results of the wave transmission coefficient are given in Figure H.9. It is clear that a larger submergence gives higher wave transmission, which is also in agreement with existing formulae given in Chapter H.1. From the figure it is also seen that even though results from many different wave conditions are shown (conditions are given in Table H.2) the scatter is fairly small indicating that the influence of the investigated parameters is small.

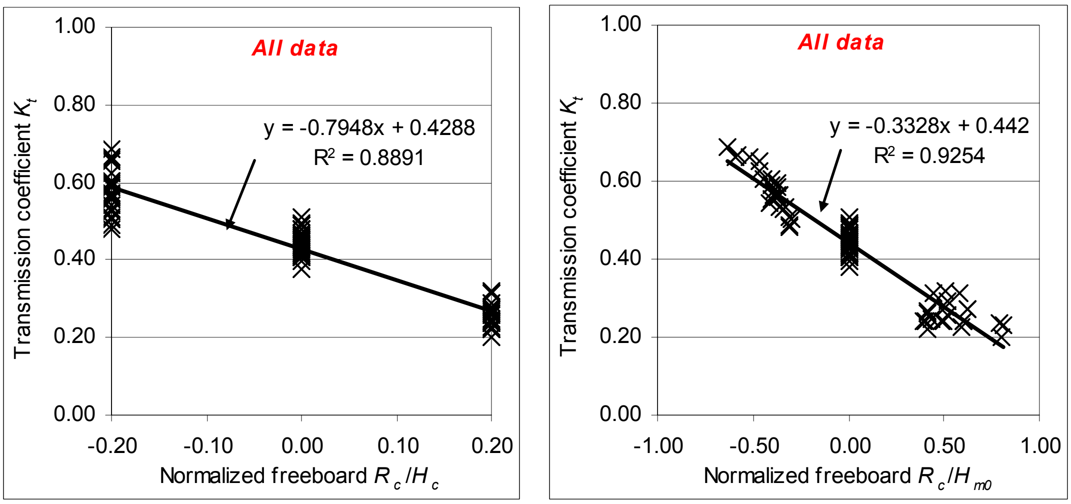


Figure H.9. All results for the wave transmission coefficient with rubble structure. Left: Freeboard normalized with structure height. Right: Freeboard normalized with wave height.

Tests with 2D waves and 3D waves were completed for the zero freeboard conditions. Results are given on the left graph in Figure H.10. It is seen that the results are in line indicating that the wave transmission coefficient is only marginally affected by the short-crestedness of the waves. From the right graph in Figure H.10 it is seen that the mean wave direction of the transmitted wave is slightly smaller than the incident wave direction. In the tests the transmitted wave direction was about 90 % of the incoming wave direction, presumably due to refraction.

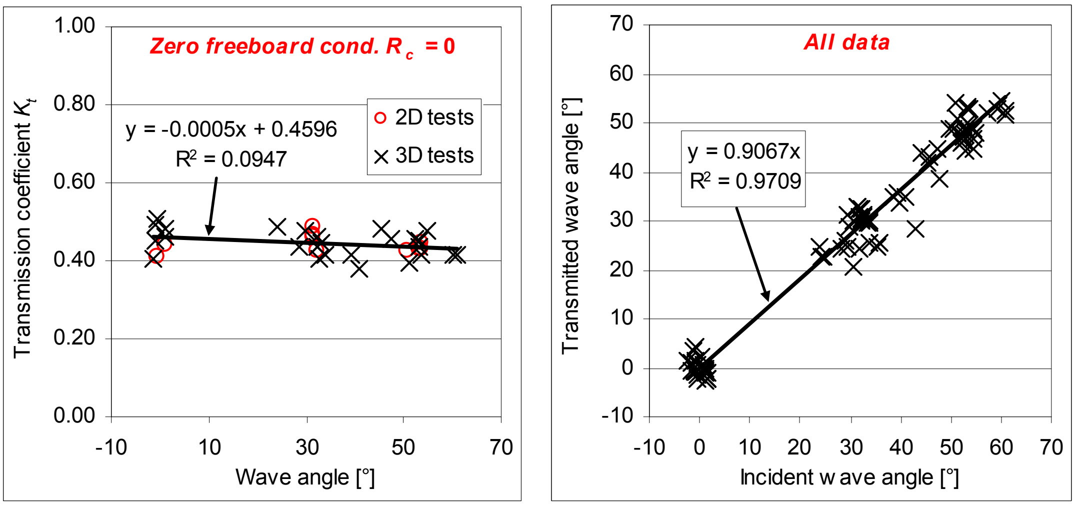


Figure H.10. Wave transmission results with respect to directionality of the waves. Left: 2D waves compared to 3D waves. Right: Incident versus transmitted angle.

The influence of the wave direction on the transmission coefficient depending on wave steepness and freeboard is investigated in Figure H.11. First of all the results generally follows the straight trendlines, with some larger scatter present for the submerged conditions. This is caused by the larger waves, which are more difficult to generate correctly in the laboratory. The scatter is about the same for both datasets no matter of the tested wave steepness', indicating that the influence of the wave steepness is not affecting the conclusions no matter of the wave direction nor the freeboard. By looking at the straight trendlines in Figure H.10 (left) and Figure H.11 (left and right), it is clear that the results shows a tendency that more oblique waves causes a slight reduction in the transmission coefficient. Further it is seen that this reduction gets more pronounced for higher freeboard, i.e. the slopes of the straight lines numerically increases when the freeboard is increased.

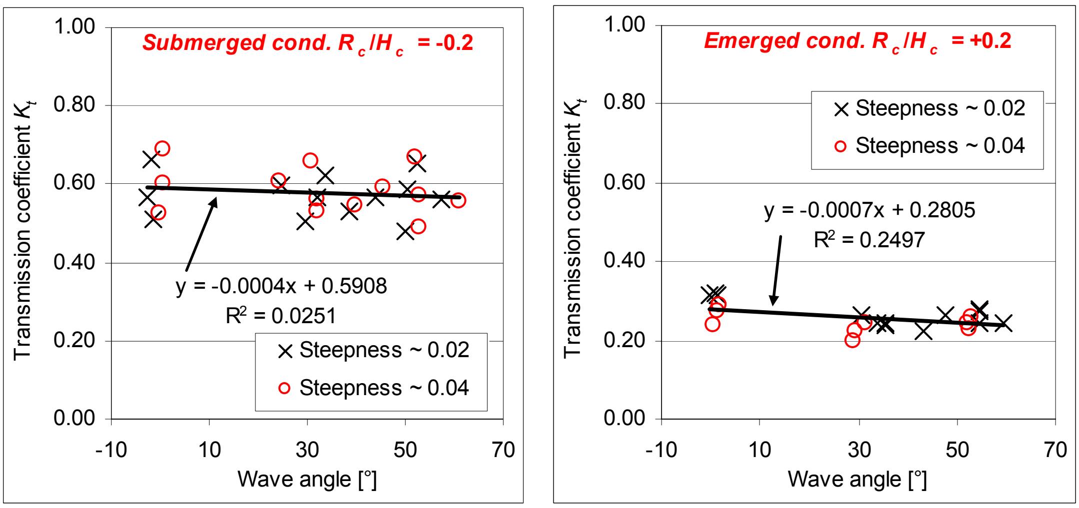


Figure H.11. wave transmission results with respect to wave direction and wave steepness. Left: Tests on submerged structure. Right: Tests on emerged structure.

In conclusions, directionality and 3D effects on the transmission coefficient for low-crested rubble mound structures are marginal.

H.2.4 Tabulated data for tests on the rubble structure

In the table "JW 3D" is irregular short-crested waves generated from the Jowswap spectrum, "JW 2D" is irregular long-crested waves generated from the Jonswap spectrum. Incoming significant wave heights H_{m0} and the mean direction is calculated by the BDM method.

Test	Layout	Freeboard	Water depth	Wave	Peak period	Seaward		Leeward		s_{op}	R_c/H$_c$	R_c/H_{m0}	H_{m0}/h	K_t
no.		R_c [m]	h [m]	type	T_p [s]	H_{m0} [m]	Dir. [°]	H_{m0} [m]	Dir. [°]	-	-	-	-	H_{lee} / H_{sea}
1	1 (gap)	0.00	0.25	JW 3D	1.60	0.097	-1	0.048	0	0.024	0.00	0.00	0.39	0.49
2	1 (gap)	0.00	0.25	JW 3D	1.88	0.122	-1	0.055	1	0.022	0.00	0.00	0.49	0.45
3	1 (gap)	0.00	0.25	JW 3D	2.12	0.140	-1	0.057	1	0.020	0.00	0.00	0.56	0.41
4	1 (gap)	0.00	0.25	JW 3D	1.13	0.065	-1	0.033	-2	0.033	0.00	0.00	0.26	0.51
5	1 (gap)	0.00	0.25	JW 3D	1.33	0.097	1	0.047	0	0.035	0.00	0.00	0.39	0.48
6	1 (gap)	0.00	0.25	JW 3D	1.50	0.116	1	0.054	-3	0.033	0.00	0.00	0.46	0.46
7	1 (gap)	0.00	0.25	JW 2D	2.12	0.142	-1	0.058	4	0.020	0.00	0.00	0.57	0.41
8	1 (gap)	0.00	0.25	JW 2D	1.50	0.130	1	0.057	0	0.037	0.00	0.00	0.52	0.44
9	1 (gap)	0.05	0.20	JW 3D	1.50	0.086	1	0.027	-2	0.024	0.20	0.58	0.43	0.31
10	1 (gap)	0.05	0.20	JW 3D	1.70	0.099	1	0.032	-1	0.022	0.20	0.50	0.50	0.32
11	1 (gap)	0.05	0.20	JW 3D	1.88	0.111	0	0.035	-1	0.020	0.20	0.45	0.56	0.31
12	1 (gap)	0.05	0.20	JW 3D	1.06	0.063	1	0.015	0	0.036	0.20	0.79	0.31	0.24
13	1 (gap)	0.05	0.20	JW 3D	1.20	0.080	2	0.022	-1	0.036	0.20	0.62	0.40	0.27
14	1 (gap)	0.05	0.20	JW 3D	1.33	0.097	2	0.028	-2	0.035	0.20	0.52	0.48	0.29
15	1 (gap)	-0.05	0.30	JW 3D	1.70	0.096	-2	0.064	0	0.021	-0.20	-0.52	0.32	0.66
16	1 (gap)	-0.05	0.30	JW 3D	2.04	0.135	-3	0.076	1	0.021	-0.20	-0.37	0.45	0.56
17	1 (gap)	-0.05	0.30	JW 3D	2.33	0.162	-1	0.083	3	0.019	-0.20	-0.31	0.54	0.51
18	1 (gap)	-0.05	0.30	JW 3D	1.20	0.079	0	0.054	2	0.035	-0.20	-0.63	0.26	0.69
19	1 (gap)	-0.05	0.30	JW 3D	1.44	0.112	1	0.067	1	0.034	-0.20	-0.45	0.37	0.60
20	1 (gap)	-0.05	0.30	JW 3D	1.65	0.149	0	0.079	0	0.035	-0.20	-0.34	0.50	0.53
21	2 (30°)	0.00	0.25	JW 3D	1.60	0.097	33	0.043	29	0.024	0.00	0.00	0.39	0.45
22	2 (30°)	0.00	0.25	JW 3D	1.88	0.130	34	0.054	30	0.024	0.00	0.00	0.52	0.41
23	2 (30°)	0.00	0.25	JW 3D	1.88	0.133	41	0.050	35	0.024	0.00	0.00	0.53	0.38
24	2 (30°)	0.00	0.25	JW 3D	1.88	0.126	28	0.055	24	0.023	0.00	0.00	0.50	0.43
25	2 (30°)	0.00	0.25	JW 3D	2.12	0.149	33	0.061	31	0.021	0.00	0.00	0.60	0.41

Test no.	Layout	Freeboard R_c [m]	Water depth h [m]	Wave type	Peak period T_p [s]	Seaward H_{m0} [m]	Seaward Dir. [°]	Leeward H_{m0} [m]	Leeward Dir. [°]	s_{op} -	R_c/H$_c$ -	R_c/H_{m0} -	H_{m0}/h -	K_t H_{lee} / H_{sea}
26	2 (30°)	0.00	0.25	JW 3D	1.13	0.073	30	0.035	29	0.037	0.00	0.00	0.29	0.48
27	2 (30°)	0.00	0.25	JW 3D	1.33	0.105	32	0.049	31	0.038	0.00	0.00	0.42	0.46
28	2 (30°)	0.00	0.25	JW 3D	1.33	0.112	39	0.047	36	0.041	0.00	0.00	0.45	0.42
29	2 (30°)	0.00	0.25	JW 3D	1.33	0.105	24	0.051	25	0.038	0.00	0.00	0.42	0.49
30	2 (30°)	0.00	0.25	JW 3D	1.50	0.134	32	0.059	30	0.038	0.00	0.00	0.53	0.44
31	2 (30°)	0.00	0.25	JW 2D	2.12	0.151	32	0.064	33	0.022	0.00	0.00	0.60	0.43
32	2 (30°)	0.00	0.25	JW 2D	1.50	0.135	32	0.062	33	0.038	0.00	0.00	0.54	0.46
33	2 (30°)	0.00	0.25	JW 2D	1.50	0.126	32	0.061	32	0.036	0.00	0.00	0.50	0.49
34	2 (30°)	0.00	0.25	JW 2D	1.50	0.135	31	0.063	33	0.038	0.00	0.00	0.54	0.47
35	2 (30°)	0.05	0.20	JW 3D	1.50	0.083	34	0.020	26	0.024	0.20	0.60	0.41	0.24
36	2 (30°)	0.05	0.20	JW 3D	1.70	0.100	36	0.024	25	0.022	0.20	0.50	0.50	0.24
37	2 (30°)	0.05	0.20	JW 3D	1.88	0.126	36	0.030	26	0.023	0.20	0.40	0.63	0.24
38	2 (30°)	0.05	0.20	JW 3D	1.88	0.120	43	0.027	28	0.022	0.20	0.42	0.60	0.22
39	2 (30°)	0.05	0.20	JW 3D	1.88	0.122	31	0.032	21	0.022	0.20	0.41	0.61	0.27
40	2 (30°)	0.05	0.20	JW 3D	1.06	0.062	29	0.012	26	0.036	0.20	0.80	0.31	0.20
41	2 (30°)	0.05	0.20	JW 3D	1.20	0.085	30	0.019	25	0.038	0.20	0.59	0.42	0.22
42	2 (30°)	0.05	0.20	JW 3D	1.33	0.102	32	0.025	24	0.037	0.20	0.49	0.51	0.24
43	2 (30°)	-0.05	0.30	JW 3D	1.70	0.107	34	0.067	30	0.024	-0.20	-0.47	0.36	0.62
44	2 (30°)	-0.05	0.30	JW 3D	2.04	0.139	32	0.079	31	0.021	-0.20	-0.36	0.46	0.57
45	2 (30°)	-0.05	0.30	JW 3D	2.04	0.139	39	0.074	35	0.021	-0.20	-0.36	0.46	0.53
46	2 (30°)	-0.05	0.30	JW 3D	2.04	0.135	25	0.081	23	0.021	-0.20	-0.37	0.45	0.60
47	2 (30°)	-0.05	0.30	JW 3D	2.33	0.168	29	0.084	31	0.020	-0.20	-0.30	0.56	0.50
48	2 (30°)	-0.05	0.30	JW 3D	1.20	0.085	31	0.056	30	0.038	-0.20	-0.59	0.28	0.66
49	2 (30°)	-0.05	0.30	JW 3D	1.44	0.126	32	0.071	30	0.039	-0.20	-0.40	0.42	0.56
50	2 (30°)	-0.05	0.30	JW 3D	1.44	0.122	40	0.066	34	0.038	-0.20	-0.41	0.41	0.54
51	2 (30°)	-0.05	0.30	JW 3D	1.44	0.125	24	0.076	23	0.039	-0.20	-0.40	0.42	0.60
52	2 (30°)	-0.05	0.30	JW 3D	1.65	0.150	32	0.079	30	0.035	-0.20	-0.33	0.50	0.53
53	3 (50°)	0.00	0.25	JW 3D	1.60	0.095	55	0.045	48	0.024	0.00	0.00	0.38	0.48
54	3 (50°)	0.00	0.25	JW 3D	1.88	0.125	53	0.054	50	0.023	0.00	0.00	0.50	0.44
55	3 (50°)	0.00	0.25	JW 3D	1.88	0.121	60	0.050	54	0.022	0.00	0.00	0.49	0.41

Test no.	Layout	Freeboard R_c [m]	Water depth h [m]	Wave type	Peak period T_p [s]	Seaward H_{m0} [m]	Seaward Dir. [°]	Leeward H_{m0} [m]	Leeward Dir. [°]	s_{op} -	R_c/H$_c$ -	R_c/H_{m0} -	H_{m0}/h -	K_t H_{lee} / H_{sea}
56	3 (50°)	0.00	0.25	JW 3D	1.88	0.127	47	0.058	45	0.023	0.00	0.00	0.51	0.46
57	3 (50°)	0.00	0.25	JW 3D	2.12	0.146	51	0.058	51	0.021	0.00	0.00	0.58	0.40
58	3 (50°)	0.00	0.25	JW 3D	1.13	0.074	52	0.034	47	0.037	0.00	0.00	0.30	0.45
59	3 (50°)	0.00	0.25	JW 3D	1.33	0.105	53	0.048	49	0.038	0.00	0.00	0.42	0.45
60	3 (50°)	0.00	0.25	JW 3D	1.33	0.103	61	0.043	52	0.037	0.00	0.00	0.41	0.41
61	3 (50°)	0.00	0.25	JW 3D	1.33	0.101	46	0.048	42	0.037	0.00	0.00	0.40	0.48
62	3 (50°)	0.00	0.25	JW 3D	1.50	0.131	54	0.055	50	0.038	0.00	0.00	0.53	0.42
63	3 (50°)	0.00	0.25	JW 2D	2.12	0.136	51	0.057	54	0.019	0.00	0.00	0.54	0.42
64	3 (50°)	0.00	0.25	JW 2D	1.50	0.135	53	0.058	53	0.039	0.00	0.00	0.54	0.43
65	3 (50°)	0.00	0.25	JW 2D	1.50	0.134	53	0.058	53	0.038	0.00	0.00	0.53	0.43
66	3 (50°)	0.00	0.25	JW 2D	1.50	0.128	54	0.057	53	0.037	0.00	0.00	0.51	0.45
67	3 (50°)	0.05	0.20	JW 3D	1.50	0.080	54	0.022	47	0.023	0.20	0.62	0.40	0.27
68	3 (50°)	0.05	0.20	JW 3D	1.70	0.099	54	0.028	45	0.022	0.20	0.50	0.50	0.28
69	3 (50°)	0.05	0.20	JW 3D	1.88	0.123	54	0.030	49	0.022	0.20	0.41	0.61	0.24
70	3 (50°)	0.05	0.20	JW 3D	1.88	0.115	59	0.028	53	0.021	0.20	0.43	0.58	0.25
71	3 (50°)	0.05	0.20	JW 3D	1.88	0.120	48	0.032	39	0.022	0.20	0.42	0.60	0.26
72	3 (50°)	0.05	0.20	JW 3D	1.06	0.062	53	0.014	48	0.035	0.20	0.81	0.31	0.23
73	3 (50°)	0.05	0.20	JW 3D	1.20	0.084	52	0.020	48	0.037	0.20	0.60	0.42	0.24
74	3 (50°)	0.05	0.20	JW 3D	1.33	0.097	53	0.025	44	0.035	0.20	0.52	0.48	0.26
75	3 (50°)	-0.05	0.30	JW 3D	1.70	0.107	53	0.070	47	0.024	-0.20	-0.47	0.36	0.65
76	3 (50°)	-0.05	0.30	JW 3D	2.04	0.133	51	0.078	49	0.020	-0.20	-0.38	0.44	0.59
77	3 (50°)	-0.05	0.30	JW 3D	2.04	0.136	57	0.076	52	0.021	-0.20	-0.37	0.45	0.56
78	3 (50°)	-0.05	0.30	JW 3D	2.04	0.139	44	0.079	44	0.021	-0.20	-0.36	0.46	0.57
79	3 (50°)	-0.05	0.30	JW 3D	2.33	0.162	50	0.078	49	0.019	-0.20	-0.31	0.54	0.48
80	3 (50°)	-0.05	0.30	JW 3D	1.20	0.086	52	0.058	46	0.038	-0.20	-0.58	0.29	0.67
81	3 (50°)	-0.05	0.30	JW 3D	1.44	0.126	53	0.072	48	0.039	-0.20	-0.40	0.42	0.57
82	3 (50°)	-0.05	0.30	JW 3D	1.44	0.123	61	0.068	52	0.038	-0.20	-0.41	0.41	0.55
83	3 (50°)	-0.05	0.30	JW 3D	1.44	0.125	46	0.074	43	0.039	-0.20	-0.40	0.42	0.59
84	3 (50°)	-0.05	0.30	JW 3D	1.65	0.161	53	0.079	49	0.038	-0.20	-0.31	0.54	0.49

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